May 17, 1934

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Engineering News-Record

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Trestle of preframed steel bents extending out nearly 1,500 ft. into the St. Lawrence River permitted the use of land equipment in sinking six piers for the Caughnawaga Bridge near Montreal

Bridging the St. Lawrence Near Montreal

Skyscraper Wind Design by a 28-Ft. Model

Sluicing Operations at the Norris Dam Quarry

Driving a Tunnel Under Air Without a Shield

Retaining-Wall Design for Fifteen-Mile Falls Dam

Inventory Record System for Ohio Highway Bridges

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In the News:

BIDS FOR THE ALL-AMERICAN CANAL, called for June 7, involve nearly 40,000,000 cu.yd. of excavation on the first 30 mi. of the route from the Colorado River westward.

THE MIDTOWN HUDSON RIVER vehicular tunnel at New York is officially under way, the celebration of groundbreaking on May 17 having been attended by the requisite number of dignitaries. Actually the contractor has been at work since the contract was let on March 20.

A MARINE TERMINAL in New York Bay, involving an expenditure of some $10,000,000, is under consideration for Bayonne, N. J. Governor Moore, of New Jersey, signed the enabling legislation May 8 over protests from Newark, Jersey City and Hoboken.

A ROUND-UP OF PWA officials from all over the country was held in Washington May 14 and 15. Policy and program matters were completely aired.

THE PRESIDENT transmitted his suggestions for a home construction and repair program to Congress on May 14. It centers entirely on financial phases with emphasis placed on federal loans by private agencies.

In This Issue:

BRIDGE-PIER CONSTRUCTION in swiftly running water always presents a serious problem to the contractor. Hand in hand with the dangers and difficulties inherent in such work is usually an expensive construction plant. Builders of the new Caughnawaga Bridge over the St. Lawrence near Montreal solved the problem with a unique preframed trestle reaching 1,500 ft. out into the stream. Ordinary land plant was used in sinking the piers and in constructing the trestle.

ON THIS SAME CAUGHNAWAGA BRIDGE the superstructure contractor erected 235-ft. deck-truss spans as cantilevers, by utilizing a special erection harness over the piers. Thus, complete and effective bypassing of the swift currents and the floating ice of the St. Lawrence was achieved on this notable bridge undertaking.

SHIELD-DRIVING has been developed to such a degree of efficiency that its adoption for subaqueous tunnels is almost a foregone conclusion. That alternative methods may also be effective is illustrated by the construction of an 8-ft.-diameter cable-duct bore under the Passaic River at Newark, N. J. Pressed-steel liner plates, carried tight against the face, gave all the security that a shield could offer and speeded up progress.

THE BASIC CONCEPTS of soil pressure on a retaining wall developed in the notable large-scale model tests at M.I.T., reported by Professor Terzaghi in a series of five papers published in these pages since February, were applied to the design of an actual retaining wall on the Fifteen-Mile Falls hydro development on the Connecticut River. The application of these scientific data to practical design is discussed in a concluding article in this issue.

EVERY BRIDGE on the Ohio state highway system is statistically described in a card system of visible-record type kept in the central office, with duplicates in the district offices. Activities of the bureau of bridges revolve about this bridge inventory. Found to be indispensable—it is, paradoxically enough, unique in state highway routine.

RECENT ECONOMIC DEVELOPMENTS have compelled a new consideration of the structural form, soundness and balance of the American city. Harland Bartholomew analyzed the modern needs of metropolitan areas before the U. S. Chamber of Commerce last week, and his address is given in abstract in this issue.

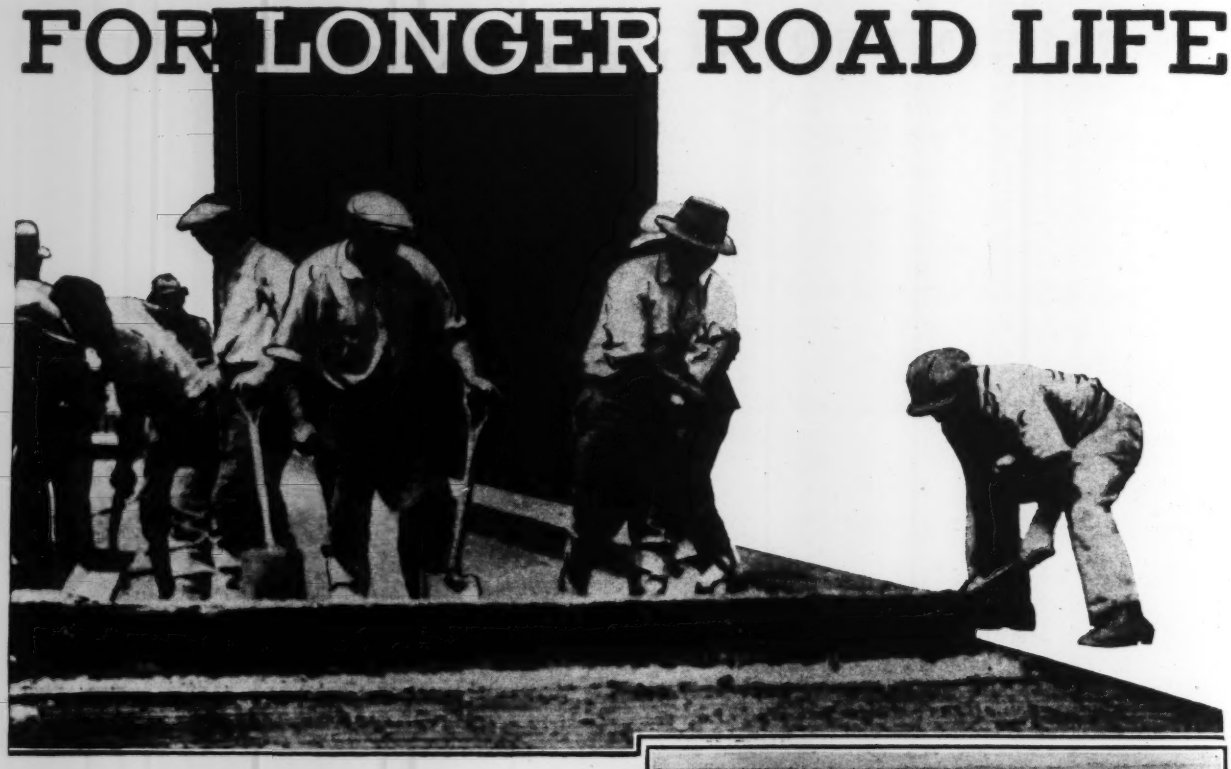
SKYSCRAPER CONSTRUCTION, when and if it is resumed, will benefit from a study of a 56-story building-bent model being conducted at Ohio State University. The model, 28 ft. high, is of steel, with three-dimensional columns. It is being used to check the Spurr theory of wind design.

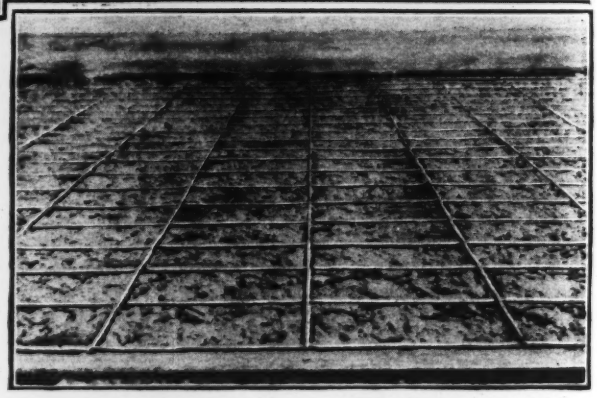
MODIFICATION of the construction program of the Chicago Sanitary District is proposed in a report submitted to the PWA in connection with an application for a loan of $42,000,000. The recommendations are summarized in this issue, together with a statement by Philip Harrington, chief engineer of the district.

Coming Articles:

A SERIES OF FOUR ARTICLES on stabilizing soil road surfaces will begin in *Engineering News-Record*, May 24. These articles summarize one of the most extensive highway research studies of recent years and undoubtedly the most complete investigation ever undertaken of the possibilities of soil stabilization for road wearing courses. Specifically the four articles will discuss theories of soil stabilization, traffic tests of stabilized trial roads, construction and maintenance methods, and costs. The articles are rich in tabulated and charted test data and in illustrations of tested roads of various soil compositions. The authors are W. R. Collings and A. C. Stewart, engineers of the road-stabilization department of The Dow Chemical Co., Midland, Mich.

FOR LONGER ROAD LIFE





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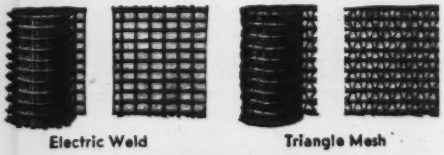

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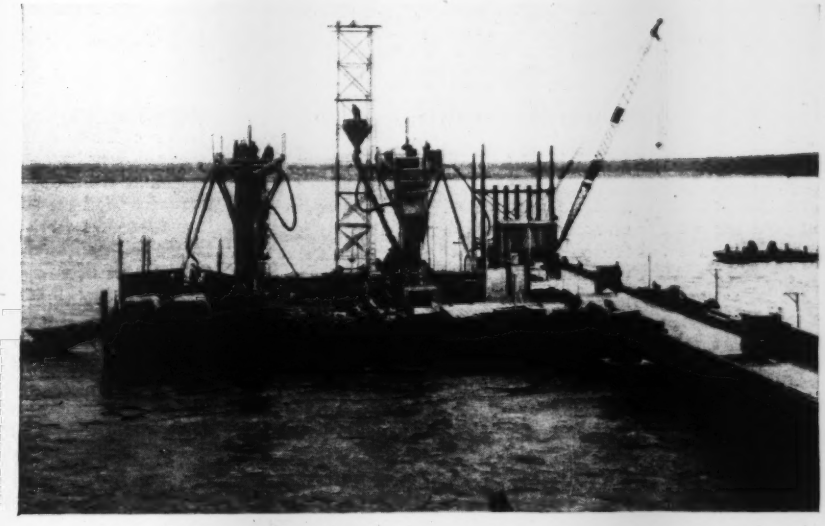


FIG. 1—SINKING A CAISSON for a new St. Lawrence River crossing in a still-water basin formed by extending the trestle around the pier site. Crawler crane, shown in background, extending trestle toward next pier location, and tractor buggies for concrete transport are major items of equipment used.

River Piers Built with Land Plant from Trestle of Unique Design

Preframed steel tower bents that support the trestle deck have hollow square posts through which timber spuds were driven into bed of St. Lawrence River in constructing the piers for Caughnawaga Bridge near Montreal

By George L. Freeman

Moran & Proctor, Consulting Engineers, New York, N. Y.

ON THE PROGRAM of the recent meeting of the Engineering Institute of Canada (*ENR*, Feb. 15, 1934, p. 242) a new highway bridge over the St. Lawrence River west of Montreal was the subject of several papers. One of these was by J. A. Lalonde, chief engineer of A. Janin & Company, Ltd., on the substructure work, and another on the superstructure was by W. Chase Thomson, structural engineer of the Lake St. Louis Bridge Corp., the public body which owns the Caughnawaga - Ville LaSalle Bridge. The following two articles have drawn upon these papers to some extent, but in addition they contain information that has developed since they were presented.

—EDITOR.

CONSTRUCTION of the Caughnawaga Bridge over the St. Lawrence River, 12 miles upstream from Montreal, was characterized by a number of interesting features, but none was more instrumental in the successful and efficient completion of the work than a special trestle used in connection with the foundation work. This trestle, about 1,500 ft. long, of preframed steel tower bents, was held in place by timber spuds driven through hollow posts of the bents. Because of its special design the trestle permitted rapid and easy installation in swiftly running water; and, by its use, ordinary and economical land equipment could be utilized in sinking the pneumatic caissons for the piers. The trestle offered a minimum amount of stream obstruction, could be dismantled easily and had almost 100 per cent salvage value.

The bridge, which was designed and constructed by the Lake St. Louis Bridge Corp., a public body, is a highway toll structure and connects the Caughnawaga Indian Reservation on the south with Ville LaSalle on the north side of the river. It spans a reach of fast water between the outlet of Lake St. Louis and the head of the Lachine Rapids. The bridge structure itself is about 4,500 ft. long, and, including approaches, is more than 6,000 ft. long. The Caughnawaga approach consists of about 1,200 ft. of earth embankment and a reinforced-concrete viaduct 1,250 ft. long, while the north approach includes about 450 ft. of em-

FIG. 2 — TRESTLE extending half-way across the St. Lawrence River, from which six piers of the new Caughnawaga Bridge were built. Gasoline buggies for hauling concrete from shore, a crawler crane and a hoist-and-chute tower comprised all of the equipment for trestle operation. At the right, equipment is shown assembled at one of the four piers built with floating plant.

bankment and a concrete viaduct 300 ft. long. The main river crossing of 2,918 ft. includes eleven deck truss spans (one of 170 ft. and the others of about 235 ft.) and one 400-ft. through-span over the navigation channel near the south shore. The second deck truss span from the north shore is designed to provide for the future installation of a lift span over the ship canal of the proposed St. Lawrence Waterway.

The river depth at the site varies from 3 to 4 ft. near the north shore to a maximum of about 30 ft. The low-water current velocity varies from around 1 m.p.h. near the north shore to about 6 m.p.h. two-thirds of the way across; and during high-water periods velocities may be as high as 8 m.p.h. Except for a channel near the south side, passing under the 400-ft. span, the river is not navigable because of the presence of many shoals, fast water and rocky reefs. The river bottom is boulder-strewn and irregular in character, with considerable sand and gravel on the north side, becoming more gravelly and rocky toward the south side. At the south shore a limestone ledge is exposed. The entire riverbed is underlain with hard shale and limestone, on which all the river piers are founded.

During the severe winters characteristic of this northern climate, there is a large amount of floating ice in the river, and occasionally the surface is frozen over except in the vicinity of the navigation channel. The site is exposed, moreover, to heavy winds and sudden squalls from the long stretch of Lake St. Louis to the west, and even during the summer months there are days when winds make it difficult or impossible to work on the river. The working season is generally from May 1 to December 1, depending on the severity of the winter and ice conditions. Extreme low water is at El. 61 and high water at El. 70.8, above mean sea level. For construction operations the river flow is fairly dependable, as changes in water levels are gradual, due to the tremendous drainage area from which the flow is derived. All of these conditions had a bearing on the selection of the trestle for foundation construction.

The general contract for the approach embankments, viaducts and river piers was awarded on Nov. 14, 1932, with completion required by November, 1934. Due to the equipment and methods adopted for the construction of the river piers this contract was completed in November, 1933, or one full year ahead of schedule.

The caisson design followed ordinary practice for pneumatic work, excepting that all steel connections above the cutting edges were welded, and the cutting edges themselves were very heavy, having a total thickness of 1¾ in., to provide against crippling while sinking through cobbles and boulder-bearing soil. The top elevation of the caissons was 60, requiring 3 or 4 ft. of light timber cofferdam to the water surface. The caissons varied from 14 to 20 ft. wide and from 60 to 74 ft. long. They were founded on solid bedrock, averaging 40 to 50 ft. below the water surface, the maximum depth being 62 ft.

The cost of furnishing and operating the necessary floating plant, the difficulty of navigating and holding floating equipment such as a compressor barge, concrete boat, material barges and tugs in the river, and the possibility of losing any such equipment which might drift into the rapids below the site led the contractor to construct piers No. 4 to No. 9 inclusive with land equipment operating from an ingeniously devised trestle that extended from the north shore to pier No. 9, a distance of about 1,500 ft.

The character of the river bottom made it impossible to drive piles and construct a trestle by ordinary methods, so a system of preframed steel trestle bents, with fender and guide cribs at each pier site, was developed which permitted short timber piles or spuds to be driven, and the framework and deck of the trestle to be constructed by a gasoline crane, projecting the work ahead from the finished structure. The function of the steel bents was to brace the timber spuds for any required depth below the water level without requiring any underwater work.

The trestle was located upstream from the piers and consisted of a plank deck 14 ft. wide and about 10 ft. above low-water level, carried on six 10x16-in. stringers supported on 12x16-in. cross-caps, spaced 20 ft. c. to c. Supporting the caps, in each bent, were four 12x12-in. square timber spuds driven into the river bottom from 1 to 5 ft. These spuds were driven through

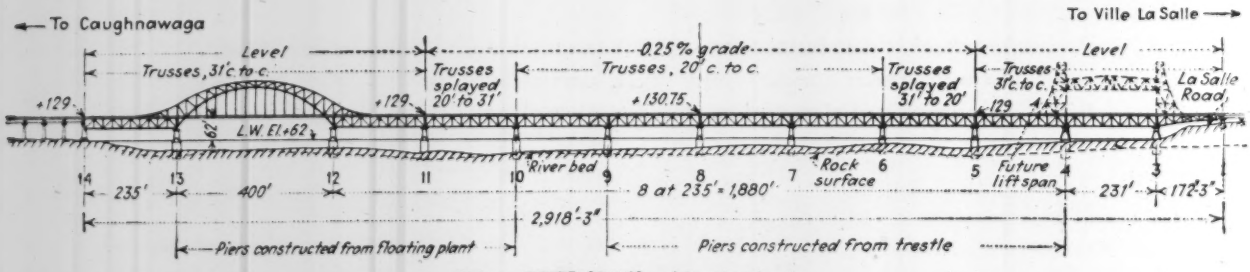


FIG. 3—RIVER SPANS and location sketch of new Caughnawaga Bridge over the St. Lawrence near Montreal. The channel span is a continuous tied arch. The future lift span will provide for the proposed St. Lawrence Waterway.

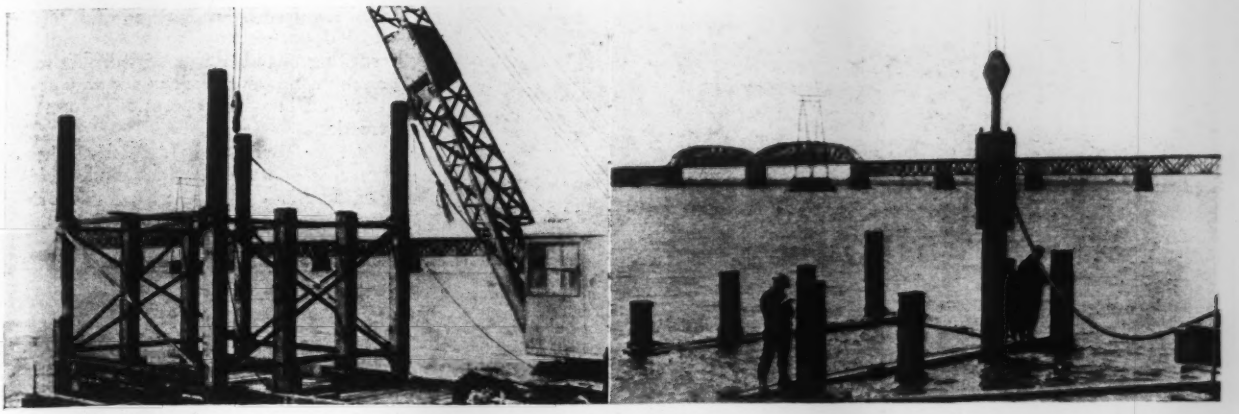


FIG. 4—TRESTLE TOWER BENTS were assembled complete (left) before being placed in water. At right, driving timber spuds that have been inserted in the hollow tube legs of the trestle frame.

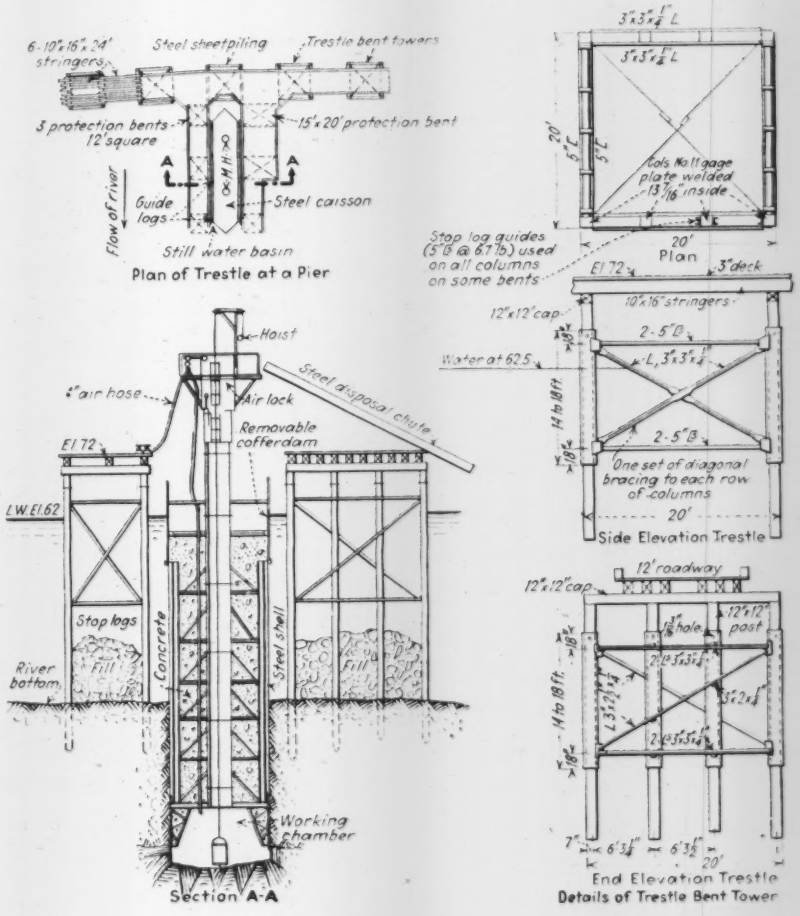
square tubes made of $\frac{1}{8}$-in. bent plate, these tubes in turn being prefframed and braced together to form towers about 20 ft. square (Figs. 4 and 5).

The construction procedure for the trestle was as follows: The steel bents, containing four pile tubes each, were delivered to the job, with necessary loose members to brace the bents into 20-ft.-square towers. The towers were assembled immediately back of the crane on the trestle, all connections being bolted. The four corner spuds were lowered through the tubes a sufficient distance so that when the crane swung the frame into position in the water, the spuds would land on the bottom with the frame substantially level. The structural frame was held in position on the spuds by means of pins near the tops of the tubes, and anchor lines were attached to the upstream side of the structural frame to hold it in position against the current. Then the two intermediate spuds in each bent were dropped through their respective tubes and driven to refusal in the river bottom, and pins driven into these spuds supported the frame while the corner spuds were released and driven to refusal. After the eight spuds were driven they were cut off at the proper level, capped, the trestle deck extended 40 ft., and the crane advanced 40 ft. ready to assemble and set the next frame.

To still the water at each pier site, and to provide means for guiding the caissons, steel tower bents, similar to the trestle bents, were arranged as shown in Fig. 5, one directly upstream from the pier and two on each side, with platforms extending on both sides of the pier to a point near the downstream end. Stop-log grooves were provided in these frames, which permitted the introduction of plank stop-logs forming closed boxes, which could be filled to provide additional stability where required. The cribs on each side of the caissons were filled with riprap to a height of 10 to 15 ft. above the river bottom. Fig. 4 shows one of the trestle frames assembled with its four corner spuds ready to be swung ahead into the water by a crane, and also this same frame with the hammer driving home the last spud, the tops of the tubes just showing above the water level.

Fig. 1 shows the entire equipment set-up required for prosecuting the river work from the trestle. The caisson is being sunk under air and is equipped with two air locks, each provided with a small jib crane and air hoist for handling the dirt buckets. Concrete is being placed in the caisson from the tower on the far side. The 1-cu.yd. gasoline concrete buggies shown on the trestle deliver transit-mixed concrete from the north shore to the tower. A crawler crane is preparing to place one of the trestle frames, which is com-

FIG. 5—DETAILS of prefframed steel tower bent trestle construction used on Caughnawaga Bridge.



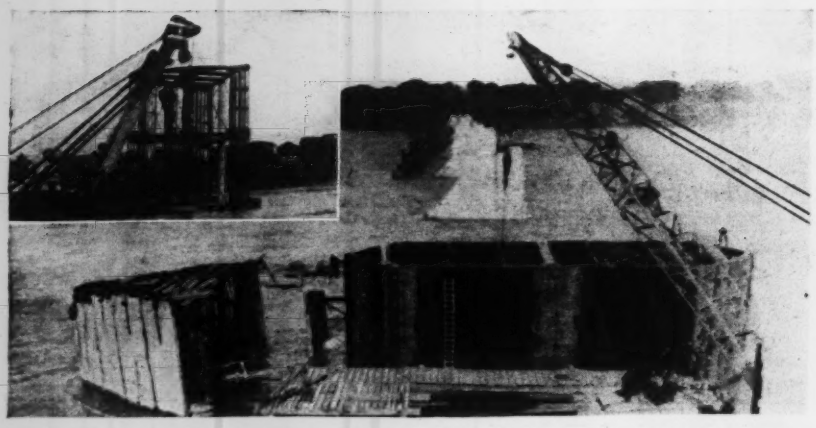


FIG. 6—BREAKWATER or deflector used to anchor caissons of the four piers constructed with floating plant. Insert shows derrick boat lowering frame of deflector into water, ready for concrete panel stop-logs.

pletely assembled with its corner spuds; and a small barge anchored upstream, equipped with two hand winches, carries light guy lines to steady the frame against the current while it is being lowered to the bottom. This view shows all the equipment used except a stiff-leg derrick on the north shore, which unloaded trucks and landed materials on the shore end of the trestle. The gasoline concrete buggies were used not only for transporting concrete, but as tractors for hauling materials from the north shore derrick to the several piers.

Steel for caissons 4, 5 and 6 was shipped on trucks and trailers from the Lachine shops of the Dominion Bridge Co. in sections that could be handled by the derrick at the north shore and the crane on the trestle. Working chambers were assembled on timbers spanning transversely across each caisson berth, after which the caissons were built to a height of 14 ft. and lowered into the water. Caisson cutting edges, decks and side walls were assembled at the shop in as large pieces as could be delivered to the job and handled by the job equipment, and practically all field connections were electrically welded.

Piers 7 to 13 were subcontracted to the Foundation Co., Ltd., of Montreal, and the subcontractor, equipped with the necessary floating equipment, built piers Nos. 10, 11, 12 and 13 from floating equipment, while Nos. 7, 8 and 9 were built from the trestle. The caissons for these piers were built up full height at the Dominion Bridge Co. shops. Caissons 7, 8 and 9 were floated on their sides to the bridge site, then lifted to an upright position by a derrick boat and loaded with concrete until they floated in an upright position, the other caissons being floated upright to the site. For guiding and holding caissons Nos. 10 to 13 inclusive the subcontractor devised a triangular structural-steel breakwater or deflector, which was sunk to the river bottom, ballasted and anchored just upstream from the caisson, while alongside these caissons guide frames were provided similar to those used for caissons 4 to 9.

The rapid progress of the job is indicated by the fact that ten piers, Nos. 4 to 13, inclusive, were constructed in five months, between May 15 and Oct. 15, 1933. Generally about two weeks was required from the time each caisson floated upright in the water until the working chamber was sealed. As many as three caissons were under way at one time during the operations from the trestle.

O. Lefebvre and J. A. Beauchemin were, respectively, chief and resident engineers for the Lake St. Louis Bridge Corp. The general contractors, A. Janin & Co., Ltd., Montreal, retained Moran & Proctor, of New York, as consulting engineers to furnish technical advice and supervision of the caisson work. H. Englander, who developed and patented the steel tube trestle design (later assigning the patent to A. Janin & Co.), was in charge of the river work for the general contractor and also was the representative of Moran & Proctor on the job. C. J. E. Maxwell was general superintendent, and W. A. Gilmore chief engineer for the contractor. The Foundation Company, Ltd., of Montreal, was represented on the job by R. B. Holland, vice-president, and **W. U. Smick**, superintendent.

Cantilever Steel Erection on Caughnawaga Bridge

Erection saddles over piers permit deck traveler to place 235-ft. deck truss spans without falsework—Arch of 400-ft. span erected in three weeks in sub-zero weather

CANTILEVER erection procedure, in several variations, characterized the superstructure work on the new Caughnawaga Bridge over the St. Lawrence River, west of Montreal. Although largely of deck truss type, the bridge is distinguished by a continuous tied arch near its south end. Special erection harness or saddles placed over the piers permitted the erection (with deck travelers) of the 235-ft. trusses without requiring the use of falsework in the swiftly running water. On the tied arch a light traveler was used in combination with the deck travelers to bring the span to closure at the center. Finally one of the anchor arms for this span was cantilevered from falsework bents. Erection in spite of cold weather and other difficulties maintained the gain in schedule time which the foundation contractor had made. The bridge is thus ready to open more than a full year ahead of the date originally planned.

The roadway on the bridge is 27 ft. wide between curbs, providing for three lines of traffic. However, the trusses on some of the spans are 20 ft. c. to c., and on others are 31 ft. c. to. c., transition between the two widths being accomplished by splaying the trusses on spans 5 and 10. This splayed design was adopted in order to facilitate uninterrupted cantilever erection from the north shore to the arch over the channel, and also to eliminate an abrupt change in width that would have detracted from the appearance of the structure. The trusses for the first three north spans (2, 3 and 4) are spaced 31 ft. c. to c., in order to provide sufficient clear width for the roadway to pass through the future lift span, which eventually may replace the present span 3. (See Fig. 2 of preceding article for an elevation view of the bridge.) The trusses for the continuous tied arch—that is, on spans 11, 12 and 13—are 31 ft. c. to c. to provide the necessary clear width for the roadway, which passes through the trusses, and also to give adequate lateral stability. The trusses for spans 6 to 9, inclusive, are spaced 20 ft. c. to c., with the floor beams resting on the top chords and projecting on either side.

Spans 11 to 13, inclusive, are con-

FIG. 1—CONTINUOUS tied arch of 400-ft. center span and 235-ft. side spans, whose closure in March marked the virtual completion of a new highway crossing of the St. Lawrence River near Montreal.

tinuous over four supports, the thrust of the arch (span 12) being resisted by a tie at floor level. Also, the curved top chords of the arch are continuous with the horizontal top chords of the anchor spans.

Erection was begun at the north end in September, 1933, spans 2 and 3 being erected on falsework so that span 3 could serve as an anchor arm for cantilever erection of span 4. The erection of span 4 was typical for the remaining deck spans. First, the permanent end jacking strut was blocked up over pier 4, and the two bottom-chord members (two panels long) cantilevered with erection shoes or kickers attached, to transmit thrust between the two adjacent spans. The bottom chords were supported by temporary tieplates between the adjacent gussets, which were replaced by top chord ties immediately after the first panel of the trusses had been erected. Truss members, laterals and floor steel for the two first panels were then erected, followed by the posts and toggle harness of the erection saddles, which were necessary to increase the effective depth of the truss over the pier for cantilever erection. The temporary top chord ties were removed as soon as the load had been transferred to the harness.

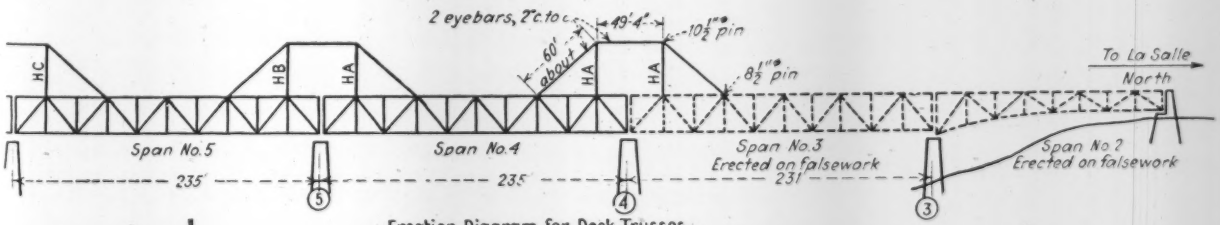
The twelve-ton-capacity traveler was then moved ahead to the second panel point and progressively to the fourth, sixth, eighth and ninth panel points in erecting the truss to a support on the jacking strut on pier 5. A follow-up derrick of 5-ton capacity placed floor steel. As soon as landed, the outer end of the span was jacked up (20 in. approximately) to release the harness at the other end. The span was then lowered with jacks to the shoes, after which the follow-up traveler dismantled the erection harness and moved it ahead to pier 5.

This saddle or erection harness (Fig. 2) consisted of portal-braced verticals supported on the top chords of the first panel points of the anchor arm and of the span being erected, horizontal ties at the top between the portal-braced verticals and inclined ties pinned to the top chords of the adjacent spans at the third panel points. Ties were double eyebars 10 in. by 1½ in.

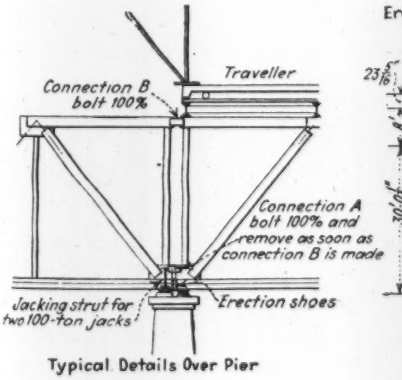
Harness width adjustable

The steel erector developed a convenient design for the erection harness (Fig. 2), which permitted its continuous use in spite of the fact that the vertical members were set variously from 20 to 31 ft. apart. This design provided for increasing the width of the harness by altering the arrangement of

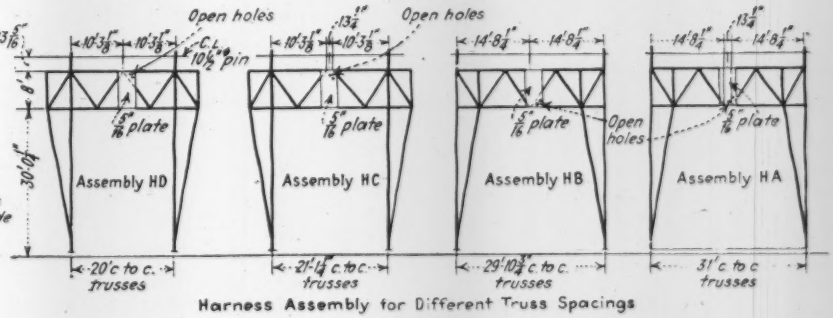
FIG. 2—CANTILEVER erection plan for 235-ft. deck truss spans of Caughnawaga Bridge involved use of special harness over piers, to eliminate need for falsework. Note arrangement for varying width of harness to accommodate it to different truss spacings.



Erection Diagram for Deck Trusses


Traveller
Connection B bolt 100%
Connection A bolt 100% and remove as soon as connection B is made
Jacking strut for two 100-ton jacks
Erection shoes

Typical Details Over Pier



Harness Assembly for Different Truss Spacings

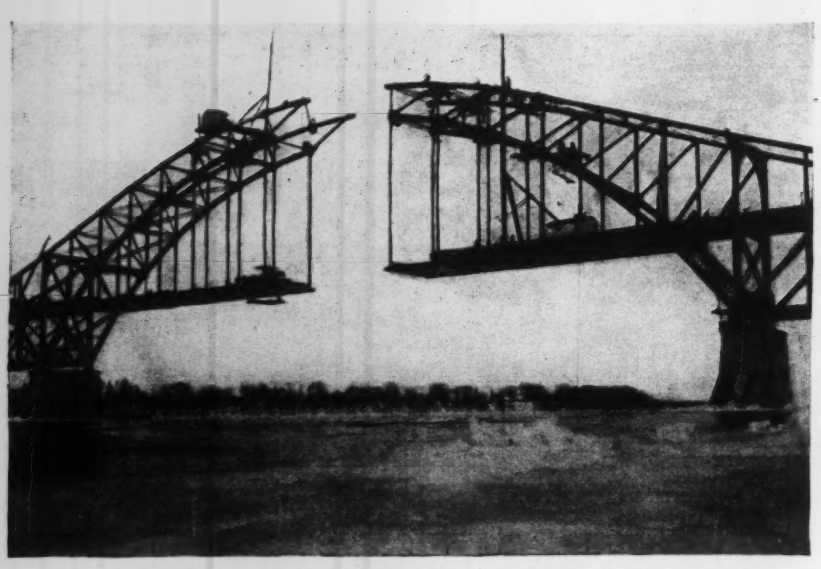


FIG. 3—ARCH ERECTION was carried out by deck traveler on the south side and a top chord creeper on the north side.

the truss panels in the portal. These panels were ingeniously attached to the vertical legs; for the narrower widths these panels were outside the legs, but for the wider widths the legs were turned, making the panels a part of the portal. A solid plate forming the center panel was also provided with two sets of holes for connecting the diagonal truss members; for the two narrower widths of the portal the plate was set so that the holes came at the top, while for the two wider widths the plate was inverted.

Erection of the remainder of the deck truss spans followed exactly the procedure described above for span 4.

In order to conform with the revised date for completion, it was necessary that span 13, the south anchor arm for the continuous tied arch, should be erected during 1933 and sufficiently early to avoid risks from floating ice. This erection was accomplished by utilizing timber falsework towers at panel points 2 and 6. Falsework increased the hazards greatly because the water was swift and deep and the season was so late that ice trouble might be expected. However, the steel was speedily placed in spite of many bad days, and no serious accident of any kind occurred.

Erecting the arch span

Erection of span 12, the arch span, began early in February. Each half was erected as a cantilever, utilizing spans 11 and 13 as anchor arms. Material was brought out on push cars operating on a service track on the deck. The first two panels at the ends, in which the heaviest members occurred, were placed by stiff-leg derricks operating on the floor of the bridge. The original plan was to have the remainder of the erection performed by light creeper travelers operating on the top chords. However, to gain speed and for greater economy, since there were two deck travelers on the job, the procedure was modified. The south, or Caughnawaga, arm was erected to within two panels of the center (the limit permitted by the strength of the top chords) by a deck traveler equipped with a 100-ft. boom. On the north arm the top chord creeper was utilized, and this equipment erected all of the north arm steel and the two remaining panels of the south arm, after the deck traveler had been removed. Closure was made at the center of the span, as originally planned.

The joint in the bottom tie was closed first, by jacking the south half of the structure, mounted on rollers at pier 13, northward about 4 in. until it met the north half, after which the tie was fully bolted. The normal center clearance of 2 in. at 60 deg. was increased due to closure being made at a temperature of 10 deg. above zero. The jacks under the free ends of the anchor arms (at piers 11 and 14) were then raised sufficiently (about 3 in.) to bring the top chords into contact, after which they were fully bolted. The anchor arm jacks were then raised until the reactions, as indicated by the gages, agreed with the calculated amounts, when the permanent shoes were inserted. Finally, all joints were riveted and the jacks removed.

Work was carried on throughout the coldest winter ever experienced in the locality, and, although considerable time was lost, not a single accident of any consequence was recorded. The contractor for both fabrication and erection was the Dominion Bridge Co., Ltd., Lachine, Quebec, the work being done under the direction of F. P. Shearwood, chief engineer, and W. P. Murray, erection manager.

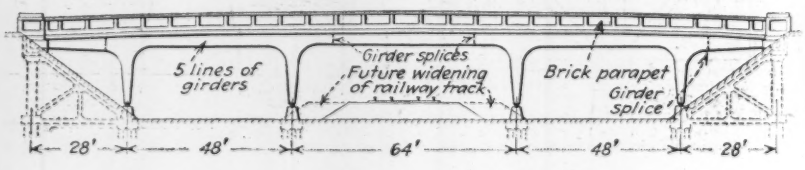
First All-Welded Highway Bridge in England Recently Completed

IN THE APPROACH to the new lift-span bridge over the Tees River at Newport, England (*ENR*, March 15, 1934, p. 365), a branch line of the London & Northeastern Railway is crossed by a five-span rigid-frame bridge, notable for being the first all-welded highway bridge in England. Poor foundation conditions complicated construction. The frame of the bridge consists of five shop-fabricated girders welded at the site to the supporting columns and plate-girder floor beams also welded into place at the site. According to an account in *The Engineer* (London), March 2, 1934, each girder is in five separate lengths, the joints allowing for expansion and at the same time simplifying the stress action. The second and fourth lengths are carried by the column legs, the third length is suspended in the center span, while the first and fifth lengths are supported between the outer ends of the second and fourth lengths and the reinforced-concrete frame abutments.

The columns are hinged at their bases, which rest on transverse reinforced-concrete beams. These beams are supported on 18-in. octagonal reinforced-concrete piles driven 70 ft. into the ground. The heels of the abutments are carried on 4-ft.-diameter concrete cylinders also sunk 70 ft.

Engineers on the work were Mott, Hay & Anderson, and the contractors, Dorman, Long & Co., Ltd.

FIRST ALL-WELDED highway bridge in England is of rigid-frame type.



Ohio Keeps a Running Inventory of All Highway Bridges

Complete and accurate data on nearly 7,000 structures of more than 10-ft. span and 57,000 culverts instantly available from visible index records

THE OHIO state highway department has available in its office at Columbus the complete history and present condition of every bridge of more than 10-ft. span on the 12,000 miles of road in the state system. This information is in the form of card-index records that have proved highly successful during the past few years when rush programs of work, to utilize relief funds, have been common. The system shows at a glance where money should be spent for repairs or where new bridges are necessary. It also gives information on the capacity of structures, useful when requests come in for routing such heavy equipment as pavers or power shovels over a certain highway. So far as the bridge department is concerned, the records constitute its "plant" inventory; and inasmuch as every change in a structure is immediately reported by the district engineers, the inventory is currently accurate.

The bridge record system had its origin about four years ago in an extensive survey of the state's bridges. The survey was undertaken with the record system in mind and was made a major order of business on the part of the bridge department. Some $30,000 was devoted to the work. Four engineering parties, covering 10 to 15 miles a day completed the survey task in about two years' time. The records have been in use since 1932 and have proved indispensable. They are probably the most important information available to the bridge department, and efficient and effective programming of work would be impossible without them. Considering its usefulness, the cost of the record system was unusually reasonable; since it is kept up to date by the district engineers in the ordinary discharge of their duties, the maintenance charges are negligible.

Every drainage structure on the state system of more than 10-ft. span is given a key number which locates it by county, by route number and by distance from the county line. All routes in the state are divided into two classes, those which run generally east and west and those which run generally north and south. The starting points, in the state, of all routes are their western or southern termini and are considered as mileage 0.0. Each structure is measured to the nearest one-hundredth of a mile from the beginning of the route. When a different county is entered the county line is taken as mileage 0.0.

In accordance with the above system of numbering, the structure known as Bridge No. Fr-40-216 would be a bridge in Franklin County on Route No. 40, 21.6 miles from the west or Franklin-Madison County line. If there were another structure in the same tenth of a mile, it would be designated as Fr-40-216A.

In case state and federal routes overlap, the structure is given the lowest federal route number, and in case of overlapping of state routes the number of the lowest state route is used.

In the filing system a separate card is devoted to each structure. These cards are duplicates of the field sheets that were used when the information was originally recorded. They set forth the dimensions of the structure, including size of members (so that the approximate strength can be computed), a sketch of the structure, a location sketch and sketches of any unusual details. A duplicate card is made for the division offices, each division thus having a record of the structures under its supervision.

The cards are filed in trays of the visible-record type (Figs. 1 and 2). In

FIG. 1—BRIDGE RECORD FILES of Ohio state highway department. In these 264 trays complete information is available on 6,856 bridges and 56,628 culverts.

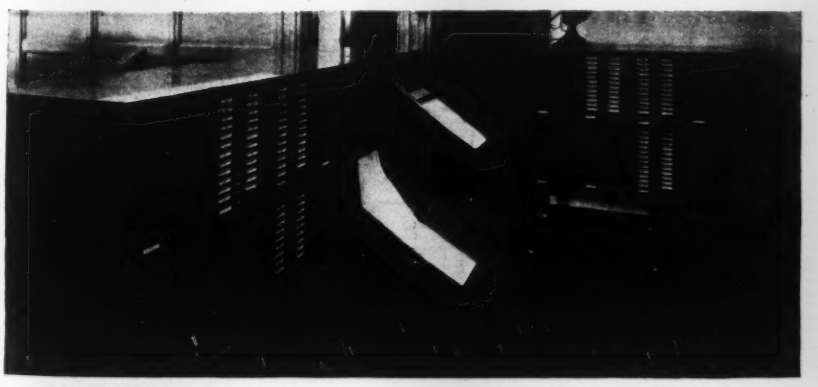
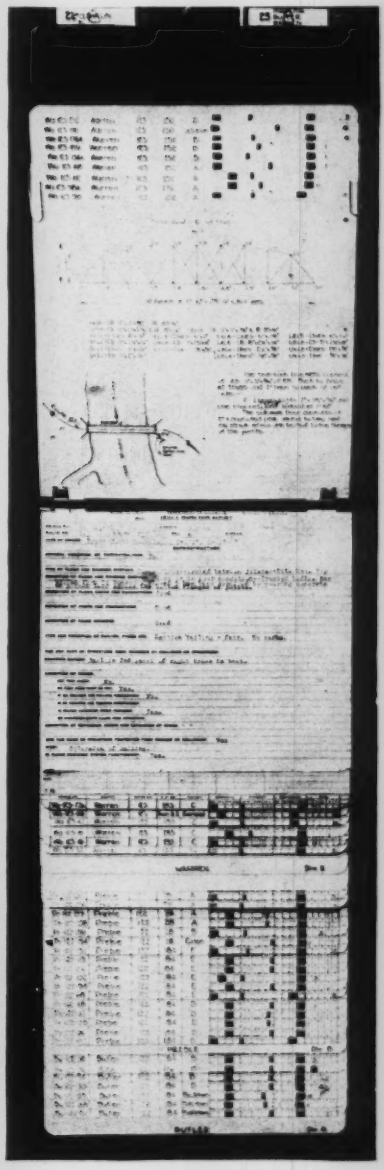


FIG. 2—A TRAY OF CARDS as it appears when pulled from its cabinet and opened up. The lower edge of each card is visible (as shown at the bottom of tray), giving an index of the principal features of each structure. The back of a card for a steel truss bridge is shown at the top of the tray, with the latest inspection report on this structure immediately beneath it.



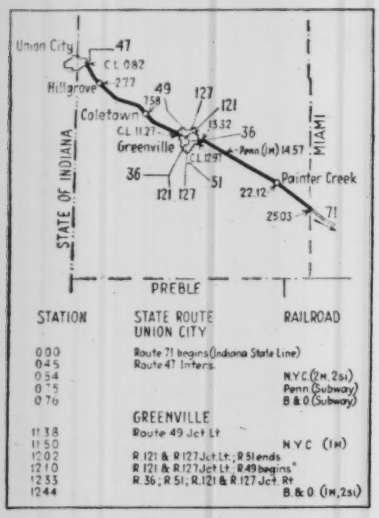


FIG. 3—MAP of state route 71 in Darke County. This map on a salmon-colored card is inserted at the beginning of the route at the Indiana state line, and a similar map card is inserted each time the route enters a new county.

the trays the routes are arranged in numerical order. At the beginning of each route and whenever the route enters a new county, a map of the route is inserted, showing its direction, the mileage of all junctions and intersections with other state routes, railroad crossings, corporation lines of municipalities and any equations necessary due to changes or relocations of parts of the route made after the original survey.

It will be noted from Fig. 2 that a small part of the lower edge of each card is visible at all times. This visible strip contains index information including bridge number, county in which bridge is located, route number, state highway number, section letter, strength classification of structure, roadway width, vertical clearance and type of structure. The letters under "type of structure" designate the following: A—arch; B—beam; G—girder; S—slab; and T—truss. The figures in the spaces under "type of structure" refer to span length, two designating spans 16 to 25 ft., three designating spans 26 to 35 ft., etc.

As a further aid to identification, different-colored cards are used to indicate the various types of structure. Thus, white cards are used for concrete and stone masonry bridges; yellow cards for steel truss bridges; buff cards for steel beam or girder bridges; brown cards for timber bridges; green cards for railroad grade-separations. Maps are placed on salmon-colored cards, and the same color card is used to separate the routes. White cards are also used for culvert information and for inspection reports, but these do not show in the visible index. The culvert cards provide space for as many as ten culverts, and are placed in the tray in their proper mileage location.

The complete filing system in the central bridge office is shown in Fig. 1. Each division is accorded two sections or 24 trays. Each tray will accommodate about 50 bridge-cards and as many culvert cards as necessary. As indicated by the accompanying table, which lists about 6,800 structures, the trays are all practically full. The cards are arranged in the file from the bottom toward the top in the same sequence that the structures occur on the highway.

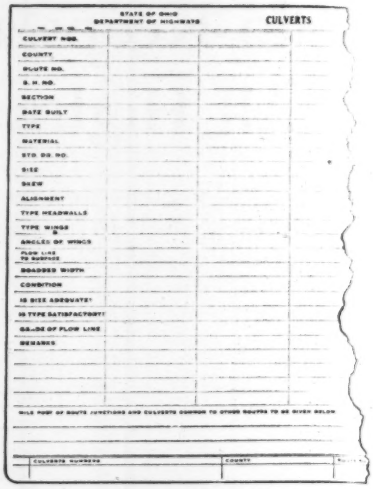
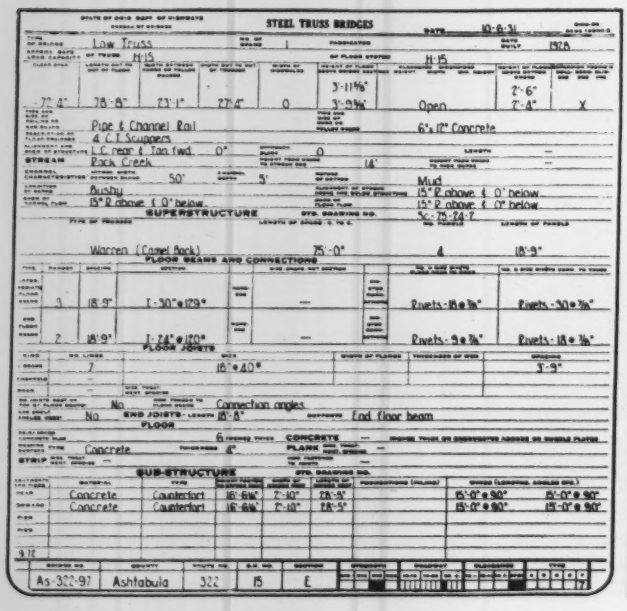
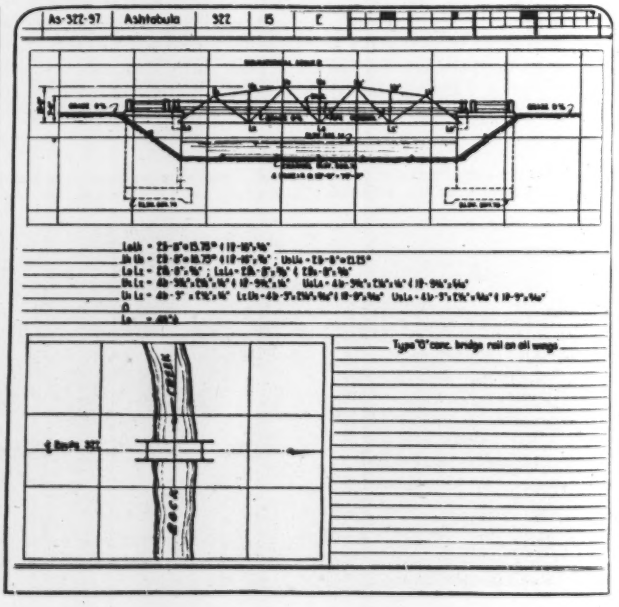


FIG. 5—CULVERT RECORD CARD which is inserted between bridge cards. It has spaces for ten culverts (five on either side of the card). It is inserted in the trays in the proper locations but is not a part of the visible record.

Fig. 2 shows the arrangement of the cards in a tray, in this case covering state route 122 and 123 in Butler, Preble and Warren counties (Division 8). At the bottom of the tray the first card inserted is salmon colored; on it is a map of the route across the county and at the bottom and always visible are the name of the county and the number of the state highway division. On top of this card is inserted a culvert card listing the culverts between the beginning of the route and the first bridge; this card is smaller than the rest and is not visible. On top of the culvert card is placed a similar size sheet carrying an inspection report for the first bridge, and this is followed by the card covering the first bridge on the route; next

FIG. 4—TYPICAL RECORD CARD, of which front and back are shown (left and right), provides complete information on a 72-ft.-span steel truss bridge. Bottom of front side and top of back side of card form an index of data that is visible at all times when the card is in the file tray. The card is yellow, while steel beam-and-girder bridges are on buff cards, concrete and stone masonry bridges on white cards, timber bridges on brown cards, and railroad grade-separations on green cards.





ALL TYPES OF BRIDGES OF STATE SYSTEM OF MORE THAN 10-FT. SPAN
(January 1, 1934)

	Clear Span in Feet					
Outside Municipalities	10–20	20–50	50–100	100–150	150+	Total
Total number each span	2,968	1,926	708	329	243	6,174
Narrow roadway (less 18 ft.)	640	525	250	124	91	1,630
Extremely weak (H-7-)	322	263	118	64	40	807
Weak (H-7 to H-12)	279	210	99	43	41	672
Both narrow and weak	449	391	189	94	69	1,192
Require replacement	792	607	278	137	103	1,917
Inside Municipalities						
Total number each span	152	182	146	62	140	682
Narrow roadway (less 18 ft.)	17	28	33	14	29	121
Extremely weak (H-7-)	14	19	22	11	15	81
Weak (H-7 to H-12)	15	25	23	6	13	82
Both narrow and weak	11	21	28	12	19	91
Require replacement	35	51	50	19	37	192
Inside and Outside Municipalities						
Total number each span	3,120	2,108	854	391	383	6,856
Narrow roadway (less 18 ft.)	657	553	283	138	120	1,751
Extremely weak (H-7-)	336	282	140	75	55	888
Weak (H-7 to H-12)	294	235	122	49	54	754
Both narrow and weak	460	412	217	106	88	1,283
Require replacement	827	658	328	156	140	2,109

follows a culvert card for the culverts between the first and second bridges, an inspection sheet for the second bridge, and then the second bridge card. This procedure is repeated until the east (or north) county line is reached. When all the cards for a given route are in place the procedure is repeated for the route with the next highest number.

In the case of structures on overlapping routes a card for the structure is placed in the file for each route but complete information is given only on the card bearing the bridge number corresponding to the lowest route number. The cards not carrying full information are marked in the lower left corner with a black quarter circle, and contains a cross-reference which locates the card bearing complete information.

A glance at a tray of cards shows all the structures, except culverts, on the route, and gives their location, load capacity, roadway width, vertical clearance, type of structure and approximate span. This is all the information necessary for the routing of large or heavy loads or for selecting structures requiring replacement, strengthening or widening.

Maintaining the records

It is only through the cooperation of the division offices that the records can be kept up to date. The central office therefore has issued detailed instructions to the division offices and made available appropriate field sheets (which are exact duplicates of the cards appearing in the index). Division offices are required to report additions, revisions and corrections that necessarily occur during any reconstruction or maintenance of the structures.

In the case of new structures, the central office submits field sheets to the division office containing all the information that can be taken from the plans. Then, inasmuch as some structures are not built in strict accordance with the plans, it is the duty of the division offices to check the central office information, add any additional information and return the field sheet to the central office with the final estimate. The central office then makes two copies of the record, one of these being returned to the division office for its file and one retained at the central office for its filing system.

A survey has recently been made of all the 2,054 railroad grade-crossings on the state highway system, and a record is now being compiled using the same system as that used for the bridge record. The visible index will portray the important characteristics of each crossing.

Personnel of survey

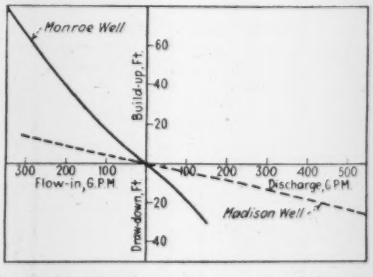
Walter G. Smith, field bridge engineer in the bureau of bridges and grade separations of the state highway department, has had immediate supervision of the survey and the development of the record system, under the general direction of J. R. Burkey, chief engineer of the bureau. O. W. Merrell is director of highways, and H. P. Chapman is assistant director and chief engineer of the department.

Well Capacity Tested by Measuring Rate Water Can Be Introduced

By John A. Oakey
Instructor in Civil Engineering, Purdue University, Lafayette, Ind.

REVERSING the usual method of testing well capacity by measuring the rate of pumping and drawdown, it is possible, and at times convenient, to introduce a measured supply of water and observe the build-up in the well. This "flow-in" method is based on the assumption that the resistance offered by the substrata to the flow of water is the same regardless of the direction of flow. The method is applicable of course only where there is an ample and convenient source of supply.

Assume that water is poured into a well at a constant rate. Under this condition, the water level in the well will build up to a level dependent upon the ability of the substrata to absorb the flow, and the head produced will be equal to the resistance offered by the substrata to the flow of water out of the well. Reversing this procedure, assume that the well is pumped at a constant rate equal to the rate of flow-in. Under this condition, the water level in the well drops to a level dependent upon the resistance offered to the flow of water in the well. It is entirely reasonable to assume, therefore, that equal rates of flow-in and discharge will produce equal build-ups and drawdowns.



COMPARISON of flow-in and discharge method of well testing.

The accompanying curves show the results of the two methods of testing on wells at Madison and Monroe, Wis. The well at Madison has a depth of 755 ft. and an average diameter of 18 in. The well at Monroe is 1,280 ft. deep with a diameter of 15 in. Both wells are cased to rock and in some sections lined to prevent caving. During the flow-in test the water was introduced by means of fire hoses leading from adjacent hydrants, with the rate measured by water meters in the hose lines. It will be noted that in some cases there is a noticeable disagreement between the build-up and the drawdown for equal rates of flow-in and discharge. Because of the conditions of the tests and the nature of this type of investigation, this discrepancy does not destroy the validity of the method outlined.

The disadvantages of the described method are the requirement of an ample and convenient water supply and a condition of the normal water level in the well which will make possible the necessary build-up. On the other hand, if these conditions exist, the method provides a rapid and inexpensive method of well testing without interfering with the drilling process or waiting for the installation of pumping equipment.

The method was suggested by Prof. L. H. Kessler, University of Wisconsin, and was used by him and the author to test several wells in southern Wisconsin.

Small Compressed-Air Tunnel Driven Without Shield

Pressed-steel liner plates carried tight against face in 8-ft. cable-duct tunnel 588 ft. long under Passaic River at Newark, N. J.—12-in. concrete lining placed by hand

A SMALL compressed-air tunnel is being successfully driven under the Passaic River at Newark, N. J., without the use of a shield. Pressed-steel liner plates are used for the outside lining, the plates being carried tight against the face, with the top segments extended ahead to form a hood when excavation is in progress. Concrete lining, placed by hand, is carried close to the heading.

The tunnel, 8 ft. in outside diameter and 588 ft. long, with a 12-in. unreinforced-concrete lining, is being driven from two shafts about 50 ft. deep. It is a power-line conduit, designed to carry 44 cable ducts. The duct lines will not be installed until the tunnel driving is completed and the air taken off. The ducts, cement-asbestos tubes, will be racked on both sides of the tunnel and completely encased in concrete, leaving an open passageway 28 in. wide through the center, as shown in Fig. 2. Twelve of the ducts, for high-tension lines, are 4½ in. in diameter; the remainder are 3½ in.

The tunnel, which is a part of the Pennsylvania Railroad's Newark improvements, skirts the north ends of four piers of a new three-track lift bridge now under construction. To keep the west shaft of the tunnel within the railroad right-of-way and still clear the west pier of the left span of the new bridge required a curve in the tunnel alignment of nearly 8 deg. near the west end. Grade of the tuunel is uniform, pitching downward 0.5 per cent eastward.

All of the bridge piers have been completed. The three river piers, all sunk by air, go down far below tunnel grade. As these piers went down, a careful check was made on the character of ground penetrated. As a result, when locating the tunnel, it was put at the elevation of the best material of the river bed, a fine yellow sand containing enough clay to give it considerable cohesion. This material has proved ideal for tunneling, and is at an elevation requiring not more than 25 lb. of air, with ample cover.

The two shafts, identical except for slight variation in depth and in top chamber construction, were sunk as pneumatic caissons. The caissons were 12x14 ft., with 24-in. reinforced-concrete walls. They were built up in 16-ft. sections and sunk without difficulty.

Tunnel driving

About three-fourths of the tunnel was driven from the west shaft. Late in the driving operations a heading was started from the east shaft, to provide a tunnel meeting rather than a tunnel and shaft meeting. Each shaft is equipped with a man lock and a material lock, both located at the top of the shaft. The man-lock shaft, 36 in. in diameter, extends to 10 ft. above mean high water.

In designing the man locks, consideration was given to the comfort and safety of the compressed-air workers. The 60-in. lock is equipped with two ring platforms, one above the other, where the men can sit down when locking in and out. Eleven men pass through the lock in comfort at one time. An electric heater inside the lock prevents chilling during decompression, especially in cold weather. The lock tender also aids in preventing chill during decompression by dropping the pressure only 2 or 3 lb. at first, then gradually reducing it to atmospheric in 15 min. (for 25 lb.) instead of reducing to half gage at first, as is the usual custom. A well-built iron ladder attached to the man-shaft is used for getting to and from the bottom, as workmen are not permitted to ride the muck buckets. In the man shaft, about 25 ft. from the top of the lock, there is another lock door. By using this second door and leaving the bottom lock door open, long timbers and rails can be locked through without difficulty.

All excavation is by hand, and no air

FIG. 1—COMPRESSED-AIR TUNNEL under Passaic River. Left: face of heading with breastboards in place during period of placing concrete lining. Steel-plate lining can be seen in the roof. Right: steel form section and concrete-placing crew. At time this photograph was taken the form was full except at crown, the crown sections being placed one at a time as arch concrete is placed.

spading is required. Muck is loaded into a small narrow-gage dump car, pushed by hand to the bottom of the shaft. The car is dumped directly into a cylindrical bucket set in a sump below track level. As fast as excavation proceeds, the steel liner plates are erected. These plates, of ¼-in. sheets, are 3.14 ft. long and 16 in. wide, eight full plates completing a ring. Horizontal joints are staggered in erecting. No calking is used.

At the start and finish of the excavation round the tunnel face is vertical, with a few breastboards bearing directly against a full ring of plates. Excavation starts at the top. As soon as possible the top segments are placed, and then excavation continues both at the top and upper sides, permitting the placement of upper quadrant plates and top plates for the next ring, forming a protecting hood. Excavation of the full face is carried forward on a slope, the plates being placed just as soon as possible.

The crews have developed a remarkable technique in shaping the tunnel outline, to allow the liner plates to be placed tight against the outside wall. Very little overbreakage of material takes place, and usually the plates are tamped slightly for a snug fit. Where an overbreakage does occur, no attempt is made to backfill. Ten rings of plates, or 13 ft., usually comprises an excavating round. At the end of the round the last ring is completed, the face squared off and the breastboards are placed.

Concrete lining

Concrete lining is carried close to the face, with never more than 25 ft. of unlined tunnel exposed at one time. A 15-ft. section of collapsible steel form is used. Under 25 lb. of air, the maximum working time allowed per shift is 6 hours.

The work is continuous for four shifts per day, six days a week. Experience has shown that in four six-hour shifts there is just enough time to drive 13 ft. of tunnel and set ten rings of liner plates, move the forms ahead and pour 13 ft. of lining. Maintaining this schedule permits concrete to be placed in the daylight hours.

Concrete is mixed in a ½-yd. mixer at a plant located directly over the west shaft. From the mixer the concrete is chuted directly into a 12-in. pipe extending down the shaft outside both the man and material locks. A flap valve near the top and another one at the bottom of the pipe serve as a lock. When charging the pipe, the top valve is open; the bottom valve is closed and fastened with a catch. The pipe holds a full charge of the mixer. When all of a batch is within the pipe, the top valve is closed, and the bottom one is opened, the weight of the concrete just about balances the air pressure on the bottom valve. The concrete discharges from the pipe into a hopper, whence it is transferred to an end-dump car. This car is pushed by hand to the form, where the concrete is dumped onto a platform within the form.

Side plates of the form are built up progressively as concreting progresses, to permit easy shoveling by hand into the invert and side walls. As the side walls fill up, the form plates are inserted until the crown section is reached. Filling in of the crown is done by shoveling from the end of a form section. A 2x4-in. piece is used to push the concrete back as far as possible as it is shoveled onto the crown form plate. These plates are set one at a time as the arch fills up.

An electric vibrator of the internal type is used in the invert and side walls, but not in the arch. As it is impossible to set a tightly filled arch by hand-shoveling, grout holes are left in the crown section for filling later. The top liner-plate segments also have grout holes, permitting the grout to flow around the tunnel outside the steel lining.

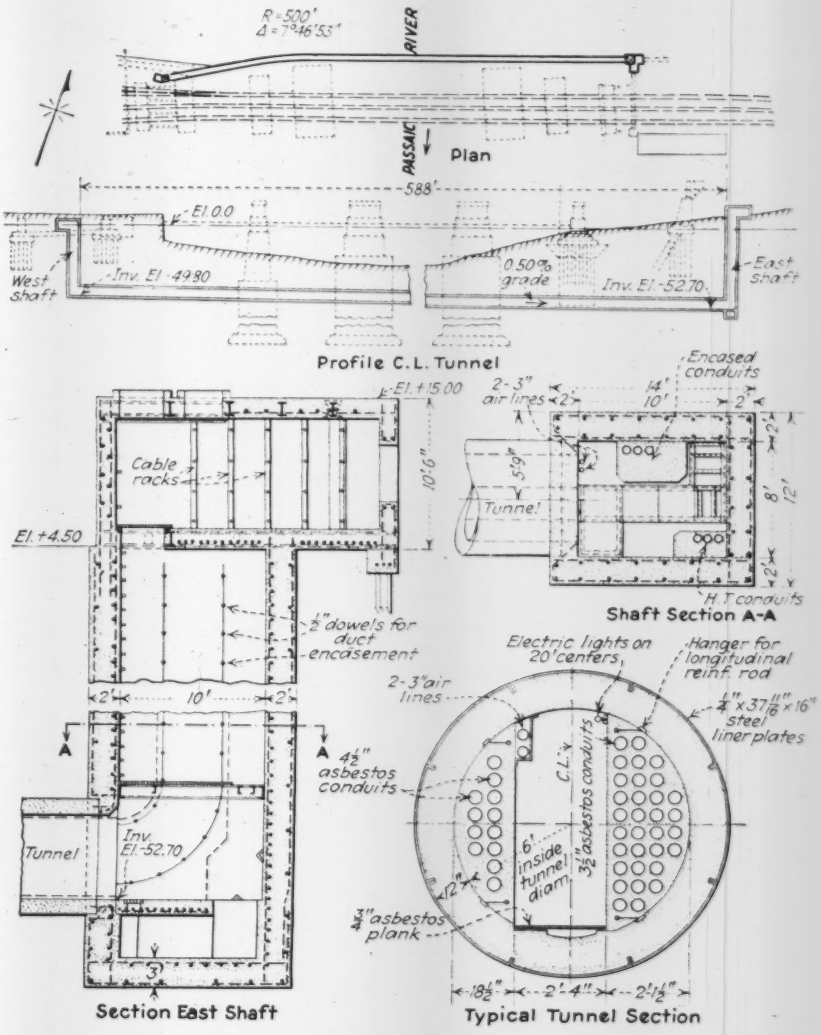


FIG. 2—DETAILS of tunnel and shaft. Plan and profile (above) show relation of tunnel to new railroad bridge, the latter shown in dotted outline. Duct lines will not be installed until tunnel driving is completed and air is taken off.

Organization

The tunnel is part of the Newark improvements of the Pennsylvania Railroad and is being built under the direction of A. C. Watson, chief engineer, New York zone; T. W. Pinard, engineer of bridges and buildings; and L. P. Struble, engineer of construction. Charles S. Landers, New York, is consulting engineer on the tunnel and bridge foundations. J. Rich Steers, Inc., New York, has the contract for much of the improvement work and sublet the tunnel to Senior & Palmer, Inc., New York.

James F. Armstrong is superintendent in charge of the tunnel construction, and John Malcolm is superintendent for J. Rich Steers.

Retaining-Wall Design for Fifteen-Mile Falls Dam

Application of large retaining-wall tests and discussion of results in designing the wall 900 ft. long and 170 ft. high separating concrete and embankment parts for large Connecticut River dam

By Karl Terzaghi
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IN A PRECEDING SERIES of articles (*ENR*, Feb. 1 and 22, March 8 and 29 and April 19, 1934), large-scale earth-pressure tests were described which essentially served the purpose of developing basic concepts for the design of the Fifteen-Mile Falls retaining wall. The following paper is intended to serve as an example of the application of the test results to such a practical problem.

Layout of wall

The wall forms the boundary between a concrete storage dam, about 1,400 ft. long, and an adjoining earth embankment with a length of about 520 ft. A full description of the power plant was published by C. R. Bliss in the September, 1932, *Journal* of the Boston Society of Civil Engineers. The wall is approximately perpendicular to the axis of the dam. It has a maximum height of 170 ft. and a length of about 900 ft. Figs. 1 (a) and 1 (b) present a front view of the wall, including a cross-section of the adjoining part of the massive concrete dam (dark shaded area). Beyond the retaining wall the storage dam consists of an earth embankment subject to percolation in a direction parallel to the face of the wall. Due to the double purpose of the wall (retaining both the earth of the embankment and the seepage water flowing through the voids of the earth) the most severe combination of forces on the different sections of the wall occurs under very different conditions.

Under normal operating conditions, the left (upstream) wing of the wall receives water pressure from both sides, and the weight of the embankment retained by the wall is reduced by the hydrostatic uplift. The elevation of the pond level varies during the year between the limits of 650 and 610 ft., and the 24-hour drawdown in practice does not exceed 6 ft. Yet, according to specifications, the possibility of a rapid drawdown from 650 to 535 ft. had to be considered (case A). In such an event the saturation line in the embankment descends much more slowly than the pond level, and as a result the retaining wall receives an unbalanced water pressure from the rear.

In the central part the earth is retained by the end of the concrete intake, which provides a large excess stability.

On the right or downstream part of the wall, adjoining the tailrace, the greatest forces act during normal operating conditions, with the storage pond entirely filled (case B, highest position of the line of saturation within the embankment).

Embankment conditions

According to the original design the embankment was supposed to consist entirely of boulder clay or similar feebly permeable material. In (a) of Fig. 1, the line ab_2d shows the line of saturation for the embankment for the case A and b_1d for the case B. Those parts of the wall which are likely, under the most unfavorable conditions, to be subject to unbalanced water pressure from the rear are marked by vertical shading.

In Fig. 1 (a) the line ab_2d of saturation almost coincides with the crest of the wall. From previous experience we knew that full water pressure would act on at least a considerable part of the back of the wall, located below the line of saturation ab_2 and b_2d respectively. Therefore, following the suggestion of the writer, it was decided prior to the tests to modify the design of the embankment and to adopt the profile shown in Fig. 1 (b).

According to this modified design, the embankment consists of a feebly permeable core b_1cc_1 of boulder clay, with slopes of 1:1. Both slopes of this core are covered by a permeable fill of sand and gravel. (After the writer left the United States in October, 1933, some minor modifications were introduced into the design of the embankment. Since this paper deals exclusively with the fundamental problems involved in the design, the modifications will be disregarded.) In order to make sure that the permeability of the outer parts would be at least twenty times greater than that of the core, the permeability of the materials delivered from the borrowpits was periodically tested by means of field tests. The permeability of the material for the core had to be smaller than 10^{-4} cm. per sec., and that for the outer parts greater than 20 times this limit. For an embankment conforming to such specifications we obtain in case B (full pond) for the line of saturation the broken line b_2c_2d. For case A the most unfavorable possibility still to be considered is represented by the line b_1b_2d, both lines as marked in Fig. 1 (b).

In order to prevent percolation along the surface of contact between the core of the embankment and the rear face of the wall, three vertical ribs (not shown in the drawing) extend from the back of the wall into the core of the embankment, serving as cutoff walls. To prevent hydrostatic uplift from acting on the base of the wall, box drains were provided for immediately above the surface of bedrock, along the section c_3c_1 of the base of the wall. Expansion joints with keys are provided at intervals of about 40 ft.

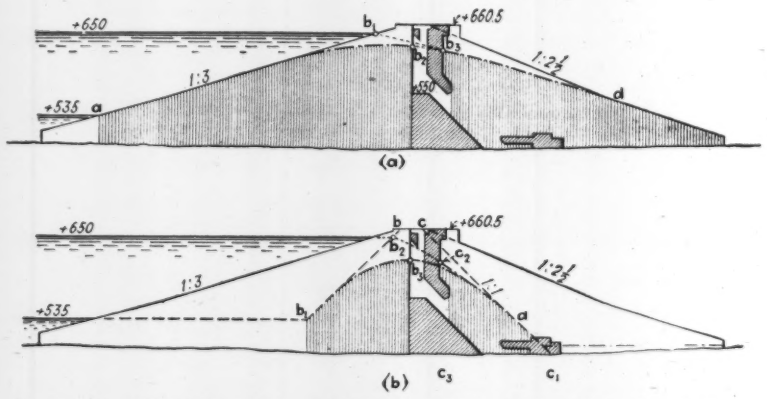


FIG. 1—TENTATIVE AND ADOPTED PROFILES for Fifteen-Mile Falls Dam on the Connecticut River.

Purpose of the tests

In connection with the design of the wall according to Fig. 1 (b) the following questions were to be answered by the tests:

1. What fraction of the full hydrostatic pressure of the water acts on the back of the wall below the line of saturation?

2. What is the effect of saturation on

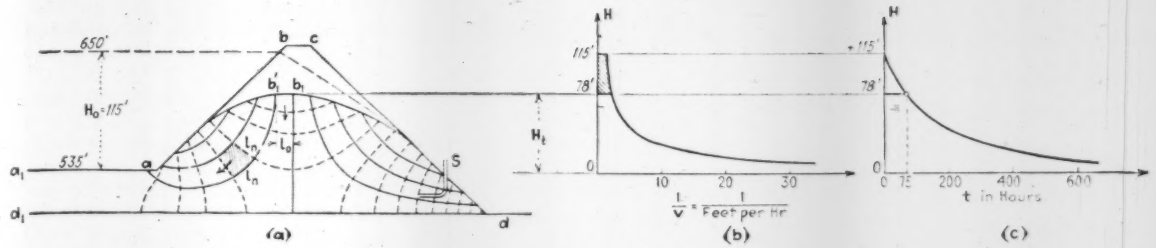


FIG. 2—SATURATION curve decline and drainage process in desaturating embankment held by retaining wall.

the angle of internal friction of the backfill?

3. What will be the position of the line of saturation immediately after the water level of the storage pond has been lowered to +535 ft?

4. To what extent does the rigidity of the wall affect the intensity of the earth pressure?

5. What are the numerical values for the coefficients of internal friction and of wall friction on which the design of the wall should be based?

6. Which of the known methods should be used for computing the forces acting on the wall?

Interpretation of test results

The answers to the questions (1) and (2) were as clean-cut as could be desired. Below the line of saturation the water acts on the wall adjoining both the permeable material and the core as though the embankment were absent. The soil weight is reduced by the full hydrostatic uplift, and the presence of the water in the voids of the fill has practically no influence on the angle of internal friction.

To answer question (3), the knowledge of three elements is required, viz., the speed with which the level of the storage pond is supposed to descend, the coefficient of permeability of the backfill, and the quantity of water which must drain out of the fill in order to establish a suction head (negative hydrostatic pressure) in the remaining part of the water content of the fill.

The speed of lowering the pond level is determined by the location of the discharge opening, by its size and by the relation between discharge and pond capacity for different heads. The assumed lowering of the pond level from 650± to 585 cannot possibly occur except through the 9-ft. disk-arm valves located at El. +535, immediately above the base of the non-overflow section of the concrete dam, discharging into the spillway channel. Owing to the limited discharge capacity of these openings, combined with the fact that the Connecticut River feeds even during low-water season at least 1,000 cu.ft. per sec. into the pond, the descent of the water level occurs very slowly. Hence question (3) was settled on the basis of a rough estimate with a very liberal allowance toward the safe side.

Yet the problem in itself is one of great practical importance and sometimes requires careful consideration. Therefore an analysis of the case will be made on the assumption that the pond is allowed to drain through the outlet channel of one of the high-pressure sluices without being fed by a river. On this assumption the level of the pond descends at a fairly uniform rate of about 0.8 ft. per hour and the emptying of the reservoir requires approximately 150 hours.

According to the test results, the transition of the boulder clay from a saturated into the drained state requires a loss of water of approximately 1.3 per cent by volume. To solve our problem we first of all compute the speed with which the crest of the line of saturation goes down, if the level of the pond were instantaneously lowered from +650± to +535. Fig. 2 (a) shows a section through the core of the embankment at an intermediate stage of this simplified drainage process. In the figure d_1d represents the surface of the bedrock (considered perfectly impermeable), a_1a the surface of the boulder-clay blanket that joins the core on the upstream side, and ab_1d is the line of saturation. The pond level is supposed to have been instantaneously lowered from 650± to the level of a_1a at +535. While the process of drainage goes on, the excess water escapes partly through the slope dc into the permeable downstream blanket and partly through baa_1 into the permeable upstream fill. Within the zone of saturation the solid curves represent the lines of flow, and the dotted curves the lines of equal hydrostatic head; that is, at any point of such a curve the water would rise in a standpipe S to the level of the point of intersection between the dotted curve and the outer surface of the core.

In accordance with the well-known laws of the flow of groundwater, the two sets of curves have the following properties: (1) they intersect each other at right angles; and (2) if we draw the curves in such a fashion that the individual fields are approximately square, the quantity of water flowing between the adjoining lines of flow is everywhere the same. Furthermore the difference of head that produces the flow from one dotted curve to the next one is everywhere the same (ΔH). The graphic procedure for plotting a set of curves of this type (isothermal family of curves) is known. (See, for example, Paul Németyi "Wasserbauliche Strömungslehre," Leipzig, 1933.

Once the set of curves has been drawn, it is an easy matter to determine the quantity of water that drains out of the core per unit of time. Let $\Delta H = 10$ ft. be the loss of head connected with the flow of water from one dotted line to the next one; $n_1 = 8$, the number of square fields through which the water passes from the crest b'_1b of the saturated space toward the surface aa_1 of exit; $k = 10^{-4}$ cm. per sec., the coefficient of permeability of the core material; $l_n =$ the horizontal distance between the points b_1 and b'_1, Fig. 2 (a), of the crest; $l_n =$ length of the side of any arbitrary square field—for instance, of the shaded field in Fig. 2 (a); $m = 0.013$, the loss of water connected with the transition of the soil from the saturated state into the drained one, as a fraction of the total volume of the soil; and $v =$ the speed at which the crest of the zone of saturation descends.

Calculating speed of drainage

According to the law of Darcy, the quantity of water which passes per unit of time through any of the square fields in the direction of the lines of flow is equal to

$$k \frac{\Delta H}{l_n} \; l_n = k \cdot \Delta H$$

Therefore, the quantity of water that escapes from the upper part of the zone of saturation, immediately below $b_1b'_1$ in Fig. 2 (a), per unit of area of the crest and per unit of time, toward the area a_1a is

$$\frac{k \; \Delta H}{l_n}$$

If we divide this quantity by m we obtain the thickness of the layer which passed from a saturated into the drained state during one unit of time. Hence the value

$$v = \frac{k}{ml_n} \Delta H \qquad (1)$$

represents the speed with which the crest of the zone of saturation

descends. Passing in Fig. 2 (a) from the crest toward either side, the values k, m and ΔH remain unchanged, while the distance between the lines of flow increases. Hence the velocity v of the descent of the line of saturation decreases toward both sides and the crest of the zone of saturation becomes flatter.

The graphic method illustrated by Fig. 2 (a) was used for computing the value v, Eq. (1), for several intermediate stages of the drainage process. In Fig. 2 (b) the abscissas represent the values $\frac{1}{v}$ and the ordinates the altitude of the crest of the zone of saturation above the final level at +535. The time t required for the crest to descend to any arbitrary elevation H_t, Fig. 2 (b), above base level is

$$t = \int_{H_1}^{H_0} \frac{1}{v}\, dH \qquad (2)$$

The value of the integral is represented by the shaded area in Fig. 2 (b). By graphic integration we obtained the diagram Fig. 2 (c), in which the ordinates indicate the elevation of the crest of the zone of saturation and the abscissas the time that elapsed since the pond was instantaneously emptied. On the assumption of drainage through the valves, the pond is drawn down in about $t_1 = 150$ hours, its level descending at approximately uniform rate. At the end of this period the crest of the zone of saturation is approximately at the same position as if the pond had been instantaneously emptied $\frac{t_1}{2} = 75$ hours previously. Hence we learn from Fig. 2 (c), that the crest of saturation descends during the drainage of the pond from its original elevation 650± to 613 ft., and the line of saturation in Fig. 2 (a) approximately corresponds to that stage.

From Eq. (1) we can derive the law of similitude whose knowledge would be required for solving the problem by a test on a reduced scale. Since the derivation of this law is a matter of elementary hydraulics, it may suffice to quote the result, illustrated by Fig. 3 (a). In this figure curve C_1 represents the relation between time (abscissas) and the water level in the storage pond. If we reproduce the full-scale embankment by a model made out of the same material but n times smaller, we must drain the small-scale storage pond in the same manner, but within the time $\frac{t_1}{n}$ (curve C_2). On this condition the following relation exists: if, at the time $\frac{t}{n}$ after the beginning of the test, the crest of the zone of saturation of the model embankment is at an elevation h above the base, the corresponding elevation on the full-sized structure will be equal to $H = nh$ at a time t.

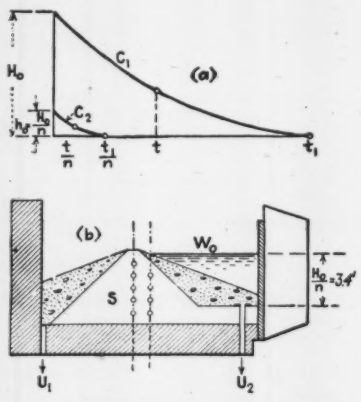


FIG. 3—METHOD for determining desaturation process by means of small-scale model tests.

The tests could easily be made in the bin that was used for the earth-pressure tests. At the outset the water level is located at W_0, Fig. 3 (b). The water percolates across the core and escapes through the outlet U_1. Then we open the outlet U_2 and drain the artificial storage pond within the time $\frac{t_1}{n}$ according to curve C_2 in Fig. 3 (a). The successive positions of the crest of the zone of saturation are determined by means of the standpipes S, which communicate with the water content of the embankment. The results of the observations should be plotted in a diagram of the type Fig. 2 (b) and Fig. 2 (c). The ordinate of the point that corresponds to the time $\frac{t_1}{n}$ represents the value h.

Effect of wall rigidity

Question (4) concerned the effect of the rigidity of the wall on the intensity of the earth pressure. This effect was considered by the writer to involve the most serious danger, because under unfavorable conditions it is likely to cause the rupture of the wall. Fig. 4 illustrates the essence of this danger. It is a section of an assumed retaining wall 46 m. (150 ft.) high, backfilled with compacted sand and gravel. The unit weight of this material is supposed to be 2.08 t per m^3, the unit weight of the concrete 2.35 t per m^3 the angle of internal friction $\phi = 34°$, and the angle of wall friction $\delta = 29°$.

According to the test results, prior to any movement of the wall ($\Delta l = 0$), the hydrostatic ratio for the backfill is greater than 0.40, corresponding to an angle of less than $\phi = 26°$ for the internal friction acting at that stage. If we compute the resultant force R_2 which acts on the base of the wall on the assumption $\phi = 26°$ and $\delta = 0$, we realize that the wall is not stable, because R_2 fails to intersect the base of the wall (Fig. 4 a). Hence the wall will yield by turning around its outer edge, whereby the lateral pressure decreases. If both the wall and the rock were perfectly rigid, the process of yielding produces a formidable concentration of pressures along the outer edge, which no concrete could stand. Therefore the wall would break as shown in the figure. On the other hand if the wall yields during the process of backfilling through a distance sufficient to mobilize the full internal friction of the fill, the resultant force R_1 would pass through a point near the outer edge of the middle third of the base and we obtain the harmless pressure distribution shown below the section Fig. 4 (b).

Hence the question arises whether or not the elasticity of the wall Fig. 4 is sufficient to eliminate the danger resulting from rigidity. According to the first of the series of articles concerning earth pressure, an average yield of the model retaining wall through a distance of approximately 0.0002 H is sufficient to mobilize the major part of the internal friction of a well-compacted backfill. According to the laws of similitude for cohesionless sand, this relation holds approximately, irrespective of the value of H. Knowing this relation, it is easy to determine the compressive stress that acts on the toe of the wall shown in Fig. 4. In Fig. 4 (c) the horizontal line a_1a_2 represents part of the base of the wall. Based on the test results for dry compacted sand, we were able to estimate the magnitude of the lateral earth pressure which corresponds to different average distances Δl through which the wall yielded. Combining these forces with the weight of the wall itself, we obtained the magnitude and the position of the resultant R which acts on the base for different average yields Δl. By plotting the values of Δl, the magnitude V of the vertical component of R and the toe pressure produced by R as ordinates above the point of application of R, we obtained the curve C_1 (values of Δl), b_1b_2 (values of the vertical component V, and C (values of toe pressure σ).

From the curves thus obtained we can draw the following conclusions: If the wall yields through an average distance of more than Δl_c, curve C_1, Fig. 4 c, the pressure on the outer edge of the wall will be less than admissible (value σ assumed to be equal to 25 kg. per sq.cm.), and the wall will have the required factor of safety. On the other hand, if it yields through less than Δl_c the wall must break because the toe pressure exceeds the compressive strength of the concrete (140 kg./cm.²). Hence the true factor of safety will depend on the degree of rigidity of the bedrock and the wall.

The computation of the real toe pressure was made on the following unfavorable assumptions: The rock on which the wall rests is perfectly rigid

and the modulus of elasticity of the concrete is 200,000 kg.per cm.² (2,800,000 lb. per sq.in.). We also neglected that part of the deflection that is due to the deformation of the upper half of the wall.

To understand the method, let us analyze the conditions which prevail if the resultant force R intersects the base of the wall at any arbitrary point a in Fig. 4 (c). In order to obtain a resultant force R which passes through a, the wall must have yielded through an average distance Δl (ordinate of curve C_1). By computing the average deflection $\Delta l'$ of the wall produced by the resultant force R and plotting the result as an ordinate above point a, we obtain one of the points of curve C_2. In general $\Delta l'$ can be either greater or smaller than Δl. The true deflection Δl_x of the wall is determined by the condition $\Delta l = \Delta l'$. Hence the unknown quantities Δl_x, V_x and σ_x are determined by the magnitude of the ordinates which pass through the point P of intersection between the two curves C_1 and C_2.

The diagrams Fig. 4 (e) and Fig. 4 (f) show the results of a similar computation made on the assumption that the wall is backfilled with dry loose sand (weight 1.98 tons per m^3.) The friction values deduced from the data contained in the first earth-pressure article are: internal friction $\phi = 32°$, wall friction $\delta = 27° 30'$. For a perfectly rigid wall they are: $\phi = 20°$, $\delta = 20°$, and after the wall has yielded through an average distance of 0.000136 H they are found to be: $\phi = 24° 30'$ and $\delta = 27° 30'$.

The diagrams Fig. 4 (c) and Fig. 4 (f) disclose the following important fact concerning both compacted and loose backfills: The concrete is compressible enough to exclude the possibility of rupture due to insufficient flexibility even of a massive concrete retaining wall. If the toe pressure is computed without considering the effect of rigidity on the earth pressure, the real toe pressure cannot possibly exceed the theoretical value by more than about 100 per cent. For the Fifteen-Mile Falls wall we could convince ourselves that the toe pressure cannot exceed the admissible value for concrete.

Question (5) concerned the values of the angles of internal friction and of wall friction to be introduced into the computation of the retaining wall. The answer to this question required a broader interpretation of the test results for two reasons. On the one hand the earth-pressure tests with compacted backfills demonstrated that the coefficient of friction that developed along the critical plane within the backfill (average normal pressure between 0.025 to 0.045 kg. per cm.²) was far in excess of the friction values obtained by friction tests under medium and high surface pressures (more than 1 kg. per cm.²).

On the other hand, the same tests showed that the high frictional stresses produced by the yielding of the wall never persisted for very long. As time passed, the stresses decreased and the coefficient of active friction approached a lower limiting value. Drainage and saturation accelerated and aggravated this condition. The lower limiting value ϕ was found to be without exception somewhat lower than the value obtained by a friction test under high pressure. The following data illustrate this conspicuous fact:

Plum Island Sand:

$\phi_{max} = 53°50'$ (1.37)	$\phi_{min} = 37°0'$ (0.75)	$\phi = 32°20'$ (0.63)
$\delta_{max} = 36°40'$ (0.74)	$\delta_{min} = 32°50'$ (0.645)	$\delta = 12°20'$ (0.22)

Boulder Clay:

$\phi_{max} = 43°30'$ (0.95)	$\phi_{min} = 32°40'$ (0.64)	$\phi = 29°0'$ (0.55)
$\delta_{max} = 33°50'$ (0.67)	$\delta_{min} = 32°40'$ (0.64)	$\delta = 16°10'$ (0.29)

Note: Values of tangent given in perentheses.

According to the writer's opinion, the two groups of facts described before—viz., the high friction values for very low pressures and the tendency of the corresponding stresses to decrease—are strictly interdependent. This opinion is based on the following considerations:

At very low pressures (sliding surface loaded to less than 0.1 kg. per cm.²) the critical shearing forces are also very small and accidental; microscopic irregularities of the surface of the grains are able to prevent further movement. Yet the equilibrium thus obtained is very unstable. Insignificant external causes such as small vibrations suffice to cause the grains to settle into more stable positions, whereby part of the intergranular stress is relieved. If water percolates out of or into the fill, the grains are subject to small but slightly unbalanced hydrostatic forces which also produce a rearrangement of the grains associated with a decrease of the internal stresses.

In contrast to this, in a mass of grains under high pressure subject to shear, minor obstacles against the grain movement are simply polished off, and as a consequence the coefficient of internal friction is lower than it is for the same material under very low pressure. At the same time the grains are held together by frictional forces of great magnitude. The unbalanced hydrostatic forces acting on the grains during drainage and saturation remain the same as they are for a mass of sand under low pressure. Hence their effect on the structure of a backfill of great depth should be small if compared to what it was found to be at the small-scale tests.

Similar considerations apply to the coefficient of wall friction. According to the test results this coefficient dropped on account of the time effect and due to drainage and saturation in the sand to 0.35 and in the boulder clay to 0.45 times the value obtained by high-pressure friction tests. Considering the great mobility of the grains in a mass

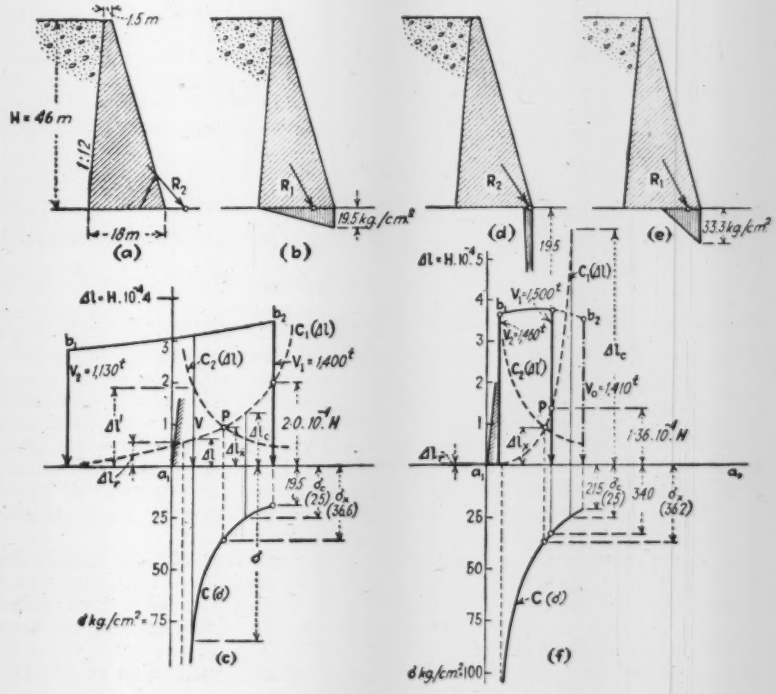


FIG. 4—EFFECT OF WALL RIGIDITY on pressure of retained earth and possible resulting hazards.

of sand under low pressure, nothing else could be expected. On the other hand, in a backfill of great depth the grains are pressed against the rough back of the wall with a great force, and as a consequence they are far less likely to change their position. Furthermore, in such a fill there is one more source of wall friction absent in a test on a comparatively small scale. It resides in the relative displacement between fill and wall due to the gradual subsidence of the fill. Therefore the writer believes that the friction between wall and fill should, for a high wall (more than 100 ft.) be more effective and permanent than for a low one.

Guided by the preceding considerations, the writer recommends for general practice the following safe values:

Coefficient of internal friction $= 0.85 \tan \varphi$
Coefficient of wall friction $= 0.35 \tan \delta$

wherein ϕ and δ denote the angles of internal and of wall friction obtained by a friction test with a surface pressure of at least 5 kg. per cm.^{2} on samples whose density is approximately equal to that of the proposed fill. In our special case the wall was to be backfilled by two different materials — viz., a natural mixture of sand and gravel (outer parts) and boulder clay (central part). For the compacted sand and gravel the data were:

$\tan \phi = \tan 42°40' = 0.92$, and
$0.85 \tan \phi = 0.78$.
$\tan \delta = \tan 33°0' = 0.65$, and
$0.35 \tan \delta = 0.23$.

Due to the urgency of the case the final values had to be selected before the earth-pressure tests were finished. The writer recommended 0.674 for the coefficient of internal friction and 0.55 for the wall friction. The first value was too low, the second one too high. The resulting error represents approximately 5 per cent on the safe side.

For the boulder clay we obtained from the tests:

$\tan \phi = \tan 32°40' = 0.64$, and
$0.85 \tan \phi = 0.55$.
$\tan \delta = \tan 32°40' = 0.64$, and
$0.35 \tan \delta = 0.22$

In the central part of the wall subject to the lateral pressure of the boulder clay rather exceptional conditions exist. The right part of this central section bears against the body of the concrete intake. The left part had to be designed to withstand an unbalanced hydrostatic pressure which acts only for a short period in the improbable case of a sudden drawdown starting with the pond completely filled. Therefore, we selected at the very outset higher values than those which correspond to normal conditions and proposed for the coefficient of internal friction 0.58 and for that of the wall friction 0.55. Since the pressure exerted by the solid fraction of the saturated boulder clay represents less than 25 per cent of the total pressure, the difference between th forces computed on the basis of the correct values (quoted above) and the assumed ones is less than 4 per cent of the total.

Method of computation

The answer to question 6 concerning the method of computation required an estimate of the relative merits and defaults of the different theories of earth pressure. The theory of Rankine was eliminated from the very start because it is at variance with the established facts concerning wall friction. The theory of Coulomb has two weak points: It assumes the sliding surface to be perfectly plane, and it gives an erroneous conception concerning both the distribution of the lateral pressure on the back of the wall and the location of the center of the pressure. However, the results of the more advanced theories of earth pressure (theory of Kötter) are almost as far from the truth revealed by the tests as are those of the simple Coulomb theory, and the error due to assuming plane sliding surfaces is small if compared with the uncertainty connected with selecting the numerical friction values. The second defect of Coulomb's theory can easily be remedied on the basis of the test results. Therefore, the writer recommended that the retaining wall be computed by means of Coulomb's theory with the modification that the center of the pressure exerted by the dry fraction of the backfill is located at an elevation of 0.42 H above the base of the wall instead of at 0.33 H.

At the time when the first design of the wall was made (fall of 1928) the opinions concerning the lateral pressure exerted by a saturated boulder clay were in a hopelessly controversial state, none of them being backed by experimental evidence. In the light of th results of the subsequent tests w realized that the layout according t Fig. 1 (b) represented important sav ings over those according to Fig. 1 (a provided both designs are based on a equal factor of safety.

The theories of soil mechanics led u to suspect that the earth pressure actin on the back of a perfectly rigid wal should be between two and three time greater than the pressure computed b means of Coulomb's theory. This ex pectation was fully verified by the te results. In 1928 nobody could say in advance how far the wall must yield in order to mobilize the internal fric tion of the backfill. The tests supplied all the data required for estimating the forces that act on a wall of limited flexibility, thus eliminating the necessity of introducing an excessive factor of safety.

The third controversial question concerned the location of the point of application of the resultant pressure. Up to 1928 no reliable tests were made with compacted backfills. According to Coulomb's theory the center should be at 0.33 H, and according to the advanced theories operating with curved sliding surfaces at about 0.36 H. The tests proved that it is located between 0.40 and 0.45 H. The erroneous theoretical values are due to the fact that the theories disregard the arching effect that precedes the first slip.

Hence the earth-pressure tests performed by the New England Power Construction Co. in cooperation with M.I.T. represent one of the most profitable investments ever made in the line of large-scale experimental work. They helped to realize important savings, eliminated the most troublesome uncertainties connected with the design of an unusual structure and furnished as a byproduct vital information on the lateral pressure of dry and saturated earth.

PWA Funds in California Spread to Include More School Reconstruction

BY A REARRANGEMENT of several PWA allotments for non-federal projects in California the sum of $3,332,200 was released and immediately re-allotted to 51 new non-federal California projects to permit a wider spread of funds in the southern California area for repair and reconstruction of schools that were damaged by the earthquake of March, 1933.

The rescinding of a $2,115,200 loan and grant made to Los Angeles County for the construction of a thirteen-story office building, cancellation of which was requested because the county had decided against construction at this time, and the rescinding of $532,000 for a sanitarium project and $342,000 for a fish harbor provided the money for the reallotments. In addition to these rescissions, a loan and grant of $460,000 allotted to San Jose for the construction of a municipal auditorium was changed to a grant of $117,000 at the city's request because it was able to finance the remainder of the project from other sources.

From these funds the PWA immediately allotted money for the 51 new projects, of which 35 are school construction or reconstruction in Los Angeles and vicinity. The remainder include five sewer projects in Los Angeles, street improvements, and one $224,000 grant to Stockton for harbor developments.

Tall Building Wind Design to Be Studied by Model

Fifty-six-story steel model 28 ft. high being built at Ohio State University to check the Spurr method of design

By George E. Large and Samuel T. Carpenter
Research Engineer and Research Fellow, Engineering Experiment Station, Ohio State University, Columbus, Ohio

LACK of a straightforward procedure for the design of continuous structures has been a great handicap to structural engineers. A design has had to be evolved by approximations, checked by one of the numerous statically indeterminate methods of analysis, "heavied up" at the weak points and then rechecked. Often these checking methods have proved so tedious and time-consuming that they have not been undertaken a second time.

So far as the wind design of tall buildings is concerned, a method devised by H. V. Spurr, chief engineer, Purdy & Henderson Co., New York, enables one to proceed logically from the very beginning to a choice of beam and column sections. The method has great possibilities for speedy and more refined designing of windbracing, and as a check on it, a steel model of a Spurr-designed 56-story three-panel bent is being prepared for test at the Ohio State University Engineering Experiment Station. A unique feature of this wind-stress model is that it is three-dimensional, the model columns being H-section as shown in Fig. 1. The model is to 1/24 size and will be about 28 ft. high when completed. It will be tested lying flat upon a long table with the lower end anchored to a concrete pier, the wind loads being applied by dead weights operating over pulleys.

The features of the steel model are so closely related to current thought and experimentation that these may logically be reviewed in discussing them.

Common design practice

In designing continuous structures, structural engineers ordinarily endeavor to visualize and evaluate the *bending deformation* (the elastic line) of the members, and proportion them accordingly. The effect of *direct deformation*, the lengthening and shortening of members, is comparatively small in most structures and is habitually ignored. In stiffly connected trusses this effect is sometimes taken into account and traditionally labeled secondary stress. Likewise, in proportioning tall building bents to resist wind, the successive floors have usually been considered as remaining horizontal under wind loading, though bending within each panel, as in Fig. 2(a), presupposing negligible direct deformation of the columns.

One such design method that has been commonly used is the so-called "cantilever method." It was described by Robins Fleming in the *Engineering News* of March 13, 1913, p. 492. (See also Mr. Fleming's book: "Wind Stresses in Buildings," John Wiley & Sons, New York.) It is based upon the following assumptions:

(a) The points of contraflexure of the columns are at their mid-heights.

(b) The points of contraflexure of the girders are at their mid-lengths.

(c) The direct stresses in the columns are directly proportional to their distances from the neutral axis of the bent.

However, in designing by this method it was customary and convenient to interpret item *(c)* as meaning total stress rather than unit stress. Then, to whatever extent the outside column was lighter than the inside one, as set by dead-load plus live-load requirements, there was an excess of direct deformation in the outer column which accumulated toward the top and produced "secondary" stresses that were never calculated.

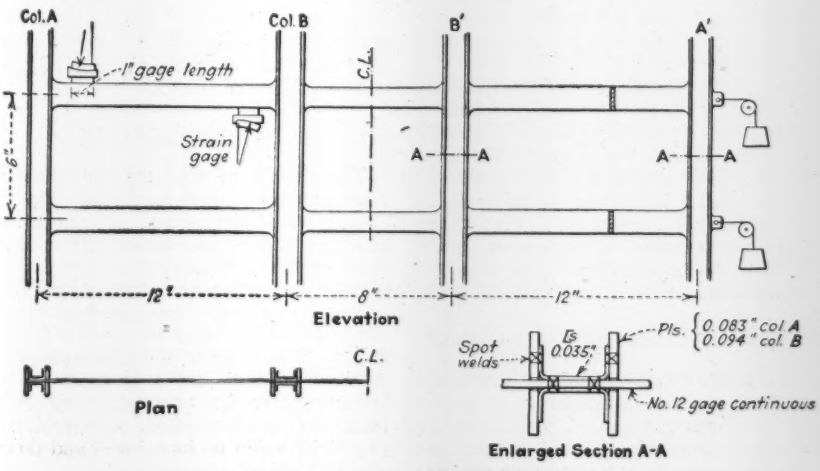


FIG. 1—TYPICAL STORIES of a 56-story steel model of a building bent designed by the Spurr method. Girders and column webs are formed by cutting openings in a flat steel sheet. Column flanges are added by spot-welding. Three-dimensional columns permit use of area and moment of inertia factors.

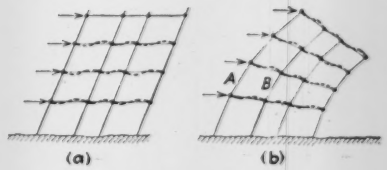


FIG. 2—COMPARISON of column and floor action under wind loads by cantilever method (a) and by Spurr method (b).

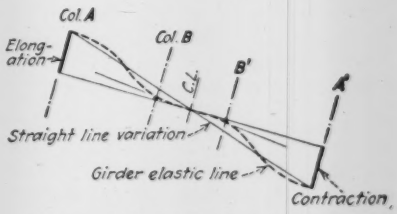


FIG. 3—POORLY proportioned bent in which excess areas of cols. B and B', over that required, result in severe bending in the girders.

Column deformations

Mr. Spurr recognizes the existence of direct deformation of columns and undertakes to prevent it from affecting the primary stresses by maintaining the tops and bases of all columns in a theoretically straight line under wind load. This means that each story must be caused to deform into a trapezoid as in Fig. 2(b), and is accomplished by contemplating a straight-line variation of column unit stress and a constant ratio of col. A and B areas in dividing wind direct stress between the cols. A and B.

Fig. 3 shows the adverse effect in an upper story of allowing the ratio of column areas to run wild. In the case shown, cols. B and B' have excess area with respect to the total direct stress delivered to them, a condition which causes extra bending in the girders similar to that produced in any continuous beam when an intermediate support settles. The true stresses in such a poorly proportioned bent can be found

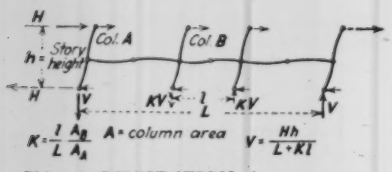


FIG. 4—DIRECT-STRESS increment per story, (V), in columns is easily determined.

only after a prodigious amount of labor. It is much easier to redesign it for straight-line variation of direct unit stress in columns. (Mr. Spurr advocates consideration of the direct deformation of columns for all buildings with height-to-base ratios greater than 5 or 6.)

As ordinarily used, the slope-deflection and Cross methods ignore the stress-producing effect of such deformations, and analyses made by two-dimensional (flat) models largely obliterate it. It is evident that the Spurr method provides a satisfactory design method and also that classic checking methods are not entirely reliable when applied to a slender, poorly proportioned bent.

Spurr method

Items (a), (b) and (c) of the cantilever method are design requirements that must be met in a Spurr design, provided item (c) is construed to refer to direct *unit* stress. With column area ratios set, (by D. L. + L. L. requirements) and the three items met, the total direct stress in each column of a lower story, due to wind, may be obtained by statics. The determination of the story increment, V, of this direct stress is even simpler, as is evident from Fig. 4. From it the division of the total story shear H, applicable to each column, can also be determined by statics. The column and girder moments are then calculated from the shears and the half-lengths. Such calculations are familiar to users of the earlier method. An example of designing by Spurr's method may be found in the report of the windbracing committee, *Proceedings*, Am.Soc.C.E., December, 1933, p. 1621; also in Mr. Spurr's book, "Wind Bracing," McGraw-Hill Book Co., New York, N. Y.

At this point the similarity of the two methods ceases. Girders were formerly chosen on the basis of section modulus, solely to meet the allowable stress, without regard to flexibility as defined by moment of inertia. Combined stresses in the columns were checked on the same basis. This procedure overlooked the necessity for having the same flexibility in each panel of a story, since the actual sway or "drift" of a story under wind must be the same in all panels. Otherwise readjustments will take place, with attendant increase in member stress or yielding at connections. The relationship of member bending and sway is shown in Fig. 5. *The theoretically correct criterion for choice of sections is*

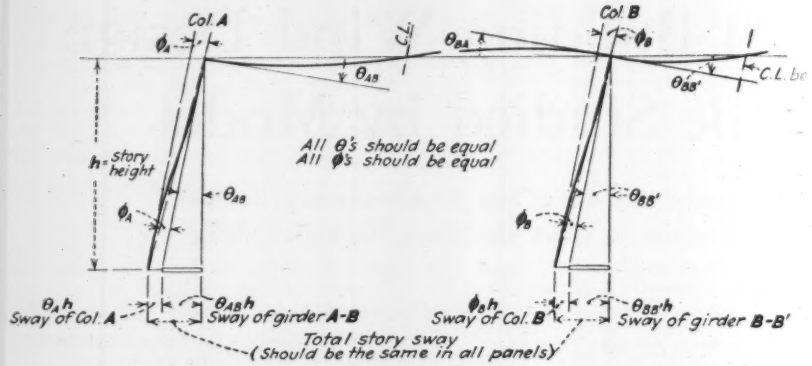


FIG. 5—RELATIONSHIP of bending and sway. The angle θ for all girders in a story should be the same, and the angle ϕ of all columns of a story should likewise be the same; i.e., the drift in every panel of a story is equal.

that the θ's of all girders in a story should be equal, and also that the φ's of all columns of a story should be equal. To satisfy this, the stresses in some members will have to run somewhat under the allowable.

Thus, in addition to providing the necessary area for a column and section modulus for a girder, Spurr also makes suitable choices of moments of inertia so that the geometric requirement of uniform deflection (sway) in all panels is substantially met.

Once some sections have been chosen, individual story designs may be checked by an adaptation of Professor Cross' moment-distribution method (*Proceedings*, Am.Soc.C.E., August, 1932, p. 1126). In its present state of development, the Cross method, great as is its utility, is unsuitable as a straightforward design procedure, since member stiffnesses must be known before it can be used.

The directness of the Spurr method has not been secured at a sacrifice of accuracy, but rather by taking advantage of the particular characteristics of the structure. It is, in fact, fundamentally more precise than the well-known academic methods because the direct deformation of the columns under wind loading is recognized. Moreover, the elastic deformation or slip of wind connections, when known or determinable, can be incorporated in the design calculations with very little difficulty. Further, the whole structure need not be treated at one time, it being quite feasible to have several designers working independently at different story levels, making choices of both columns and girders.

Questions concerning Spurr method

In designing wind bents by the Spurr method, certain questions have arisen in the minds of users. Those involving the extent to which wind connections actually contribute to sway give promise of being answered from the results of recent tests made upon actual full-sized shallow connections, both riveted and welded. Prof. W. C. Huntington, of the University of Illinois, has been engaged in this work, and so has Prof. C. R. Young, of the University of Toronto, who is about to publish a bulletin of his results. A recent theoretical treatment by U. T. Berg, "Wind Bracing Efficiency," *Transactions*, Am.Soc.C.E., Vol 98, 1933, p. 709, is of considerable interest. The question as to what is the correct effective length of a member to use in proportioning for equal sway in all panels is also involved with the performance of connections.

Questions concerning member design would seem to be answerable from the results of small-scale model studies. These arise in "designing around" structural irregularities, such as three-story entrances, assembly rooms, setbacks and basement stories. One would also like to know for certain whether the column direct stresses in a Spurr-designed bent actually do follow the prearranged straight-line variation.

Since the columns contribute a rather small proportion of the total sway, one practice is to provide merely the required column areas, without any attempt to produce equal column deflections in a story by manipulating moments of inertia. This tends to vitiate somewhat the central location of girder contraflexure points.

Use of the steel model

The results of the studies of the Spurr-designed steel model of Fig. 1 should be illuminating. Strain-gage readings will be taken, and the actual location of contraflexure points determined. Deflection measurements will be taken to check the predetermined straight-line variation of column direct stress. The sway of the structure as a whole will be observed at the various levels. After the original model has been thoroughly studied, structural irregularities will be introduced and their effect observed.

Since in this case both relative area and moment of inertia of each column had to be reproduced in the model, the

model columns could not be of rectangular cross-section. This was not the case with the girders; their direct deformation not being a factor in design, rectangular sections could be used. The vagaries of connection deformation, or "drift," had to be eliminated, necessitating an integral connection of girders to columns. These requirements have been met by machining the story openings from a large 12-gage steel sheet, providing girders and the webs of the columns. Column flanges have been made up by attaching continuous tapered channels, with coverplates, to the web by a spot-welding operation (Fig. 1).

The application of the laws of similitude to this three-dimensional model is quite simple. Linear dimensions of the actual structure have been divided by 24, the scale of the model. Areas and loads reduce with the square of 24, moments and section moduli with the cube, and the moments of inertia with the fourth power of 24. A steel model so constructed and loaded will have the same unit stresses and deformations as the actual steel wind bent.

The make-up of the model is the result of considerable preliminary experimentation. It has been found that the slender girders, due to column restraint, will not buckle laterally below the yield point of the material.

To produce theoretical deformations, a certain fillet was found necessary at the ends of girders. Specimen column sections have been made up and tested in simple bending to check the sufficiency of the spot-welding. Tests have also been made of the anchorage detail at the column bases. Some test data from the model itself will be secured before the end of the current month.

City Planning Review Shows New Factors Need Appraisal

Unplanned development continues to produce unsound municipal growth—Slum rehabilitation is sound economic and social policy

REGARDLESS of the recent emphasis placed on planning programs for rural areas and state units, there remains great need for improving the living conditions in metropolitan areas, according to statements presented by Harland Bartholomew, city planning consultant, St. Louis, in an address on "New Interests in City Planning" before the United States Chamber of Commerce at Washington, May 2. He characterized the objective of all planning as the achieving of a satisfactory standard of living for all groups of population. Recent economic developments have compelled a new consideration of the structural form, soundness and balance of the American city.

With the exception of war, the extravagance and waste in building American cities surpasses any other field of human endeavor, according to Mr. Bartholomew, and by reason of this lack of definite planning the average city has become unsound in respect to the following specific phases:

The area of expansion is far greater than is warranted by any probable growth expectancy. Public service facilities have been provided on too generous a scale, with the result that maintenance costs are an unjustifiable burden. Large metropolitan areas are allowed to disentegrate with no prospect of rehabilitation. This abandonment and the resulting shifts in population and values are costly to owners but doubly so to the city, for disease, delinquency and crime thrive in these disintegrating areas of slums where city services must be furnished at a cost two or three times larger than the taxes collected. Thus slums are subsidized. Stable metropolitan property values are unknown in America. During the last few years the point has been reached where the destruction of old values exceeds the creation of new values. No social order can long endure such a condition.

The average American city, therefore, can be classed as an extravagant and unsound structure. There is an ever-growing area of slums at the center, surrounded by larger areas of residential property in various stages of depreciation, with only a fringe of new high-value residential area near the outskirts. From 1920 to 1929 speculation in real estate undermined the structural stability of the American city and the subsequent collapse in urban property values has been as great, if not as spectacular, as the collapse in security values. Both collapses, being founded upon speculation, rather than upon social and economic need, were inevitable. However, the misfortune with respect to urban property is much the greater since the damage is physical in character and cannot be repaired quickly.

Future outlook

Mr. Bartholomew asks: What is the probable future of the American city? Because of the recognized wastefulness in methods of growth, will people abandon the cities for a more rural environment, or will people remain in the cities, adopting a new set of conditions that are frequently, but vaguely, described as a decentralized city? Will the large blighted areas of cities be abandoned, or should they be converted to parks or used for industrial development?

The history of the older European countries indicates that from 75 to 80 per cent of a total population tends to become urban. In America there is much land to occupy and develop, but despite this fact rural population declined from 1900 to 1930. The trend back to the farm from 1930-1932, resulting from the depression, was again reversed in 1933. It appears safe to predict, therefore, that our percentage of urban population will increase, possibly to 70 or 75 per cent of the total. Consequently, our cities will continue to grow, although at a very much slower rate because of the slackening of total population increase due to restricted immigration as well as to the declining birth rate.

It is doubtful if much of our population could be redistributed by attempting to build new cities. The channels of communication are vital to man's needs, and most cities are strategically located in this respect. There are very few good sites for cities not already preëmpted.

Form of future city

The simple form of the city will not be changed—a main business center, with industry located near the transportation routes and residence in adjoining areas. Where there is unplanned growth that violates unity and balanced design, there will be over-concentration of development resulting from speculative demand. As a reaction to such over-concentration there will be a general spreading of individual areas of development, particularly of business and industry, which will force expansion of residential areas. All of these developments will then tend to go to unusual and unbalanced extremes, particularly when induced by speculative demand.

"Balanced design" should therefore be the desired objective. This can be obtained only by comprehensive planning. Balanced design means an area of growth somewhat in scale with probable growth. It means establishing limits for over-concentrated development such as skyscrapers in business districts or the over-crowding of land by apartment houses. It means less subdivision of land into building lots on the outskirts of cities. It means more planning for the regulation of all these things in addition to the usual provisions for widening of streets, provision of park space or building of a civic center. Our city plans have always suggested such restrictions, but lack of public understanding and interest in such matters caused them to be ignored

in favor of piecemeal improvements fostered by speculative demands.

St. Louis, an example

Most cities lack a plan for controlling development and especially a policy which recognizes that the total area that can normally be absorbed for different uses is quite limited. As an illustration of the significance of these factors, the conditions in St. Louis County, Mo., might be cited. The city of St. Louis has a population of about 820,000 and an area of 62 square miles. It is not a congested city—in fact, it is of fairly evenly balanced design.

For the past twenty or thirty years, population has spread out into suburban areas more or less at random, until today 200,000 people are attempting to urbanize an area of possibly 240 square miles outside the present city limits. The density of population is now about one-sixteenth, although in certain small areas it is already as dense as in the central city. Several cities and towns have been incorporated, but there is much unincorporated area in which there are no building codes, fire laws, health departments or other needed services. To attempt to provide adequate public services in this great area appears to involve prohibitive costs.

A recent bond issue for roads amounted to 7½ per cent of the total assessed valuation of property. The suburban resident has temporarily secured an ample standard of open space, but the cost will sooner or later exceed that of the city. The net result of such an unbalanced process of urbanization is an increased cost for all citizens of the area and even a general lowering of the average standard of living conditions, if the population of the entire metropolitan area is considered.

Needed measures

Some of the measures now most needed to bring about social and economic stability and strength throughout the whole urban structure, according to Mr. Bartholomew, are:

Official adoption of comprehensive plans for city, county or regional government and a revision of zoning ordinances to conform to established needs are essentials and should be followed by the reconstruction of slums and rehabilitation of blighted areas. Plans are futile unless enforced. Zoning is out of step with social and economic needs. It is absurd to set aside 25, 40 or 50 per cent of the city's area for industry, commerce or tenements when at most 15 per cent is enough.

The slum is usually a logical location for residence of the low-cost housing type, being close to work centers. Reconstruction is the most logical solution for this troublesome problem. An attack should follow on the problem of blighted districts. Through properly conceived rezoning of cities, there will automatically come about a revitalization of many blighted residential areas. Where there is now only a remote prospect of resale for commercial use, a proper rezoning will indicate either the validity of a prospective commercial or industrial use, or the necessity for higher standards of maintenance based upon permanent residential use.

Housing programs

Commenting on the Federal Emergency Housing Corporation, the paper pointed out that projects should be very definitely an integral part of the comprehensive plan of the city. Local housing authorities should be created to construct, manage and operate this type of property. The buildings should be designed and equipped to provide a decent standard of living for the lower-income group of our population as distinguished from becoming merely competitive with other types of housing. Projects should return in taxes to the local municipality a sum equivalent to taxes previously collected from the areas. There is no justification for taxation of new construction as such.

The cost of normal public service, such as water, lights and the like, should also be paid by the project. Rentals should be determined upon as nearly a self-liquidating basis as possible. Since slums are already subsidized, it is only reasonable that a subsidy should be provided for new housing to replace the slums, but this subsidy should be no more than sufficient to meet the differential in cost found warranted by the economic conditions of the tenants.

Galvanized Steel Channel Forms Traffic-Lane Marker

A NOVEL, permanent traffic-lane marker, forming an integral part of a special light-weight steel and asphalt floor, has been used on the 220-ft. swing span of the new Dover Bridge over the Choptank River, near Easton, Md.

The marker consists of a heavy-weight, 3-in., galvanized-steel channel, laid toes down and arc-welded to the high channels of an interlocking steel-channel floor deck. The marker was set by transit to correct alignment and divides the 24-ft. roadway into two traffic lanes.

The steel is covered with two layers of 1½-in. mineral-surfaced asphalt bridge plank, so that the finished surface of the roadway and the channel marker are flush, offering no obstruction to scrapers used to remove snow or oil from the deck.

The steel-channel traffic marker was developed by the J. E. Greiner Co., Baltimore, consulting engineers on the movable span. The bridge was built by the Maryland State Roads Commission, of which H. O. Williar is chief engineer, and W. L. Hopkins bridge engineer.

PERMANENT traffic-lane marker furnished by a galvanized-steel channel. Floor surface is asphalt plank laid on an interlocking steel channel deck.

Modified Program Proposed for Chicago Sewage Treatment

Special Board of Engineers, appointed by Chicago Sanitary District under agreement with PWA, favors activated-sludge process for Chicago conditions—Rearrangement of Supreme Court Master's program recommended to give greater purification at earlier date

HOLDING that a modified construction program of sewage treatment for the Sanitary District of Chicago will speed the time when a greater proportion of the sewage of the district can receive a high degree of purification, the PWA Special Board of Engineers, on April 30, transmitted to Public Works Administrator Harold L. Ickes the recommendations given in brief in our news columns last week. Essentially, the program laid down in these recommendations accepts activated sludge as the proper method of disposal for Chicago conditions for all excepting an existing Imhoff installation at the West Side plant. The Public Works Administration, subject to the findings of this board, has provided funds to build an activated-sludge plant to treat the effluent from the West Side Imhoff tanks, but the board recommends that the Southwest Side plant, which must treat waste from the stockyards and packing houses, be built instead. There was no question as to the activated-sludge process being proper at the latter plant, but the board felt that something else could be done with the Imhoff effluent, possibly chemical treatment with or without filtration. Decision on this latter point is left until further study can be given to the subject.

The present status of the four major elements of the program of the Sanitary District of Chicago is as follows:

(*a*) The North Side area has a 200-m.g.d. sewage-treatment plant.

(*b*) The Calumet area will have a 136-m.g.d. plant upon completion of work now in progress.

(*c*) In the West Side area the main interceptor is partly completed, and the rest is under contract, the pumping station built, two batteries of Imhoff tanks completed, and a third under contract. This plant will provide partial treatment for 500 m.g.d.

(d) In the Southwest Side area no work has yet been done on main interceptors or treatment and pumping works.

Two combined loans and grants totaling $42,000,000 were approved by the PWA last year. The first was for $8,000,000 to complete contract work suspended on February, 1932, for lack of funds. It covered the pump and blower house at Calumet, two batteries of contact tanks at Calumet, completion of a battery of Imhoff tanks at the West Side plant and two sections of West Side intercepting-sewer tunnels. The second allotment was $33,948,000. It covered four sections of the West Side intercepting sewer, completion of the Calumet treatment works, construction of four of the Calumet intercepting sewers, three additional sections of the West Side intercepting sewer, construction of the pumping station, blower house, two batteries of aeration tanks and necessary connecting conduits for the West Side and Southwest Side sewage-treatment works. As one of the conditions upon which the loan was made, the PWA required that the Sanitary District include in its 1934 budget an item to cover the cost of a review of is whole program by a board of engineers. The board was appointed early this year. Extracts from its report, either in full or in condensed form, are given below.

The existing West Side sewage-treatment plant consists of a pumping station of 500-m.g.d. average capacity (with peak capacity 50 per cent greater), skimming tanks, grit chambers, screens, three batteries of Imhoff tanks (two batteries of which are completed and in service, while the third is under contract and 55 per cent completed), together with drying beds for dewatering of sludge and transportation facilities for its disposal. This West Side plant, as so far constructed, is for preliminary treatment only and is referred to in the reports of the Sanitary District of Chicago as providing 33⅓ per cent treatment. When designed, it was expected to supplement at some future time this preliminary treatment by the installation of a trickling-filter plant of equal capacity, 500 m.g.d.

The West-Southwest project of the Sanitary District, as now formulated, contemplates the eventual construction of an activated-sludge plant with a capacity of 1,000 m.g.d. designed to give so-called complete treatment to sewage from the Southwest Side and final treatment by the same process to the Imhoff tank effluent, the two liquids being mixed in a mixing chamber before passing to the aeration tanks and ultimate disposal of the activated sludge by incineration.

The items under review are parts of the proposed 1,000-m.g.d. plant. They include aeration and final settling tanks of 500-m.g.d. capacity, a blower and sludge pump house for the 1,000-m.g.d. plant with installed machinery for 1,000 m.g.d., and a pumping station with an installed capacity of 1,400 m.g.d. (including spares) to lift the Southwest sewage from the Southwest interceptor to the treatment plant, and to supplement the West Side pumping plant.

The plan of the Sanitary District for the treatment of West-Southwest sewage provides for merging the sewage flow from the Southwest Side with the effluent from the West Side Imhoff tanks, and treating the combined flow by the activated-sludge process. If the works now built and building for the treatment of West Side sewage were not in existence, the plan of merging the two flows in a single treatment plant would result in substantial economies. As an alternative, the board has given consideration to a plan whereby the Southwest sewage and the Imhoff tank effluent would be treated separately so that greater advantage, if possible, might be taken of the existing West Side plant.

The relative costs of treating sewage from the Southwest area by various processes are as follows per million gallons daily: activated sluge, $13.16; sedimentation-trickling filters, $11.85; sedimentation pre-aeration trickling filters, $12.31; chemical precipitation, $18.36. The disadvantages of the trickling filter combinations (large site, odor and flies) outweigh the savings in cost.

The best results that can be obtained from chemical precipitation are such that some supplemental treatment must be provided to produce the degree of purification required for the Southwest sewage-treatment works. As between the activated-sludge process and chemical processes, the cost comparisons clearly support the use of the activated-sludge process for the treatment of Southwest Side sewage. It is not equally clear to the board that the activated-sludge process is the best method of supplementing the treatment in the West Side Imhoff tanks. The data available do not indicate that there would be any saving in air by reason of this preliminary sedimentation.

Some of the plans presented to the board for supplementing the Imhoff tank treatment by chemical processes followed by filtration contemplate the application of the chemicals to the influent and others to the effluent. Evidence before the board is conflicting as to the effect of adding them to the influent, some witnesses alleging that such action would retard digestion in the tanks, while Prof. A. M. Buswell stated that the addition of chemical

coagulants, with proper dosage, to the influent of the Imhoff tanks would greatly increase their efficiency and not interfere with digestion.

In the opinion of the board, tests and experiments heretofore made should be supplemented by large-scale tests at the West Side plant to determine whether some economical combination of Imhoff tank treatment and chemical treatment, either with or without filtration, can be found which will so improve the quality of the effluent as to make further treatment unnecessary.

Regarding capacity, the board recommends reduction of the proposed Sanitary District 500-m.g.d. Southwest Side plant to 400-m.g.d. average capacity with an installed capacity for 600 m.g.d., providing 50 per cent overload. These capacities are predicated on the assumption that the Southwest area will be metered by 1940. Such a program makes no provision for growth of population and industry in the West and Southwest areas after 1940. Under ordinary circumstances, such scant provision for future growth would be unwarranted, but in Chicago the consumption and waste of water is so large that substantial reduction through metering seems highly probable. If these hopes are realized, an increase in population will be offset by a decreased volume of sewage per capita.

The items in the PWA loan contract are all parts of a combined West-Southwest plant, but so selected that upon completion the only useful work that could be accomplished immediately by the expenditure of the $21,646,000 would be to provide supplemental treatment for the already partly treated West Side sewage.

Whatever the reasons for adopting this program, they do not appear to the board to be applicable at the present time. Based on actual savings in the letting of contracts for West Side sewers and for the Calumet plant, and upon corresponding reductions in the unit costs used by the Sanitary District when preparing the construction items for its contract with the United States, it is now possible to formulate a construction program that will include all structures needed for pumping and treating Southwest sewage (400 m.g.d.) and the Southwest interceptor, including the part serving the packing-house district.

The present installation of Imhoff tanks, including battery C, now under contract, will be sufficient partially to clarify the entire sewage of the West Side; it would seem desirable, therefore, that this installation be retained for the present to carry on its work as designed, and that the new plant be utilized to purify completely the sewage of the Southwest District that will otherwise flow in its raw state to the drainage canal.

No provision is made in the new estimate for sludge disposal. Disposal by incineration, if it can be accomplished at reasonable cost, is highly desirable, and the experiments in progress are of great interest. They should, however, be tested by continuous operation over a period of months before any large-scale installation of this character is undertaken.

Experimental operation of this sort will supply information as to cost and dependability and enable a comparison to be made between this method of disposal and disposal by other methods, such as separate sludge-digestion tanks followed by filtration on vacuum filters and wastage of the filter cake on sludge dumps.

Water waste

The extravagant waste of water from the Chicago water system, which has long been known to exist and which seems to be constantly increasing, has added and will continue to add millions of dollars to the cost of the sewers and sewage-disposal plants of the Sanitary District unless such waste is materially reduced. The water annually pumped at the present time exceeds 1,000 m.g.d. and is almost double the amount that is reasonably necessary for domestic and commercial use. In many cases, water is not available even on the second floor of buildings. Pressure is not adequate for satisfactory fire service, and at the same time millions of gallons of unnecessary water are flowing unused to the sewers. For many years the attention of the city has been frequently called to the fact that only by metering can the enormous waste of water by private consumers be prevented.

The highly polluted flood discharge of the Calumet River is a serious menage to the purity of the water supply of the Southwest district. This supply is obtained from cribs near 68th St., which are only about 3 miles from the mouth of the Calumet River. There is no immediate remedy for this pollution, and the only effective method of protecting the purity of the water supply for the Southwest district is by the filtration process.

In order to eliminate unnecessary cost in construction and in the operation of such filters, waste prevention must receive early attention in this district. A consequent early reduction in the amount of sewage from this district may be expected.

The board has been greatly impressed by the large per capita consumption of water in Chicago and the consequent effect on the volume of sewage. It not only recommends that every effort be made to reduce such waste, but that the capacity of all future sewage works be predicated upon a reasonable use of water.

When the work is complete, for which PWA funds ($42,000,000) are now provided, the following results will be achieved:

DEGREE OF TREATMENT UNDER PROPOSED PLAN

District	1940 Population	Per Cent Purification
North Side	1,315,000	85*
Calumet	465,000	85*
Southwest Side	1,245,000	85*
Packing House Equiv.	1,000,000	85*
West Side	1,790,000	33
Total	5,815,000	

*These percentages assume that by 1938 means of sludge disposal will be provided.

Considering the district as a whole, this is equivalent to a weighted average of 72 per cent. If the percentage of purification of the West Side Imhoff tank effluent can be increased from 33 to 70 per cent, the weighted average for the district will be 85 per cent, which represents a reasonable degree of purification as found by the Master in his report to the U. S. Supreme Court (October term, 1929).

Engineering board personnel

As constituted, the Special Board of Engineers consisted of Daniel W. Mead, consulting hydraulic engineer; W. B. Storey, retired president of the Santa Fe Railway; Brig.-Gen. C. W. Kutz, retired army officer; and Joshua D'Esposito, Illinois state PWA engineer. Mr. Mead was chairman. The board conferred and advised with the following consultants: Harrison P. Eddy, George W. Fuller, John H. Gregory, and A. M. Buswell. The board built up a staff consisting of G. Gale Dixon, Youngstown, Ohio; S. W. Freese, Fort Worth, Tex.; E. F. Robinson, Syracuse, N. Y.; and C. W. Post, Chicago. Conferences were held with representatives of commercial processes. Full cooperation was rendered by Philip Harrington, chief engineer, Sanitary District of Chicago, and his staff, and special studies were made by the district engineers.

District's engineers supported

The following comment on the report of the Special Board of Engineers was made by Philip Harrington, chief engineer of the Sanitary District of Chicago, following publication of the board's report:

"Without having had time to study or analyze the report in detail, its general conclusions agree with and confirm the previous decisions and recommendations of the Sanitary District's engineers as to future treatment-plant policy.

"The recommendation that for the present, treatment of sewage from the West Side area be limited to sedimentation only, and that some degree of treatment be given to all sewage that affects the diversion problem, to be supplemented later by complete treatment when possible, agrees with the original policy of the district's engineers as adopted prior to the court's decree.

"The recommendation that the capacity of future treatment plants, as far as possible, be predicated upon a more reasonable use of water by the city of

Chicago, and urging a reduction in the per-capita use of water, agrees with the position always taken by the district's engineers.

"The recommendation that the Southwest treatment and pumping station be designed, having in mind the possibility that the water used from the tributary area may be metered, is also in agreement with the district's policy.

"The report finds that the Sanitary District engineers plan for treating sewage at future plants by the activated-sludge process is in accord with the best modern practice.

"The report further finds that, although lower in first cost, the cost of operation plus fixed charges make chemical processes as now developed not applicable as primary methods of purification of Chicago sewage, which likewise confirms the findings of the district's engineers.

"The report makes no definite recommendation as to method of sludge disposal, but states that disposal by incineration, if it can be accomplished at reasonable cost, is highly desirable, and that the experiments now being conducted by the district's engineers should be continued.

"The board found that high construction standards prevail in the Sanitary District treatment plants heretofore built, and that features of design which have been found to be unnecessary have been eliminated by the Sanitary District engineers by changes in design in the Calumet treatment works already under construction.

"In general, the report appears to have been made with fairness and with an appreciation of the complexities involved in the problem, and it is reassuring to the district's engineers to have an independent cooperative board of engineers of this caliber confirm the policies they have been pursuing in an undertaking of such magnitude, involving as it does the expenditure of large sums of the public moneys."

Sluicing Giant Being Used to Expose Quarry at Norris Dam

ROCK for concrete aggregates at Norris Dam on the Clinch River in Tennessee is to be quarried from a hillside just above the dam. As this hillside is steep and the overburden thin, sluicing is being employed to remove the soil and transport it down into the valley below the quarry level.

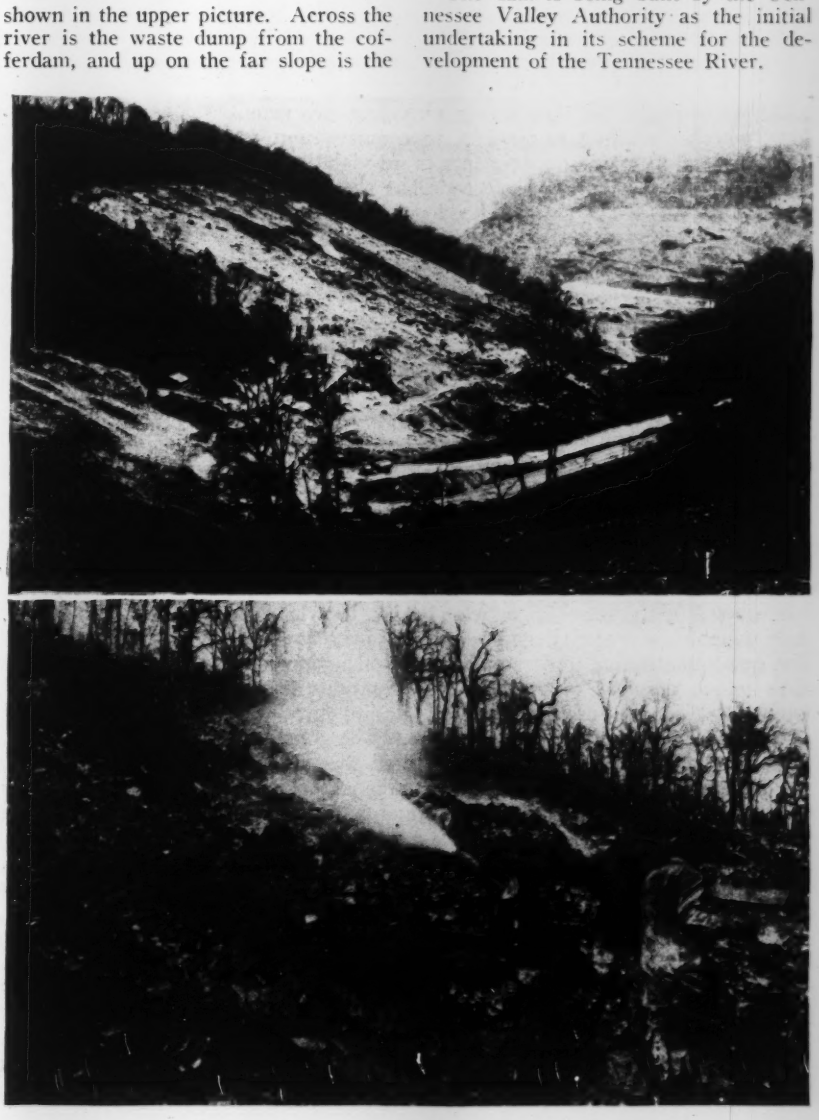
The accompanying photographs show the valley in which the quarry is located, with the sluicing operation in progress near the center of the cleared area; also a close-up of the sluicing giant. The rock is a limestone, deeply eroded at the surface, as may be seen in the close-up photograph, but it is sound farther down. Both sand and stone are to be made from the quarried rock.

The jet used in the sluicing operation has a diameter of 2½ in., and water pressure at the nozzle overages 150 lb. per sq.in. The sluiced material is carried down into the valley, where a small dam has been built to form a pool in which the heavier material settles out before the water is discharged into the river. The crest of this dam is carried up as required to maintain an adequate pool.

In the quarrying operation the rock is to be delivered to a primary crusher located on the slope beyond the temporary crusher shown in the photograph. From the primary crusher the rock will be conveyed across the side valley to a secondary crusher to be located on the slope shown at the extreme left in the upper picture, whence it will be conveyed along the side of the valley to the concreting plant located high above the dam and close to the construction cableway.

Norris Dam is under construction just below the mouth of the valley shown in the upper picture. Across the river is the waste dump from the cofferdam, and up on the far slope is the grading for one end of the traveling cableway spanning the dam site.

The dam is being built by the Tennessee Valley Authority as the initial undertaking in its scheme for the development of the Tennessee River.



SLUICING is being employed to uncover the quarry at Norris Dam, shown in the upper picture. Below is the hydraulic giant working on the quarry face.

ENGINEERING NEWS-RECORD

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Simplicity for Safety

NEXT WEEK a model traffic signal and control code is to be considered by the National Conference on Street and Highway Safety, with a view to reducing the high record of traffic accidents. Simplicity is the key to the success of this code, for simplicity is what traffic control lacks today. The effort to secure recognition of a universal code, continued now during a number of years, has consistently failed, and further continued failure is likely unless a simple and easily understood set of regulations is adopted, a code that every community in the nation can use as the basis of its control methods. Optional provisions with respect to the meaning of the color indications in automatic signals would be obviously fatal to the uniformity that is the first essential to safety; the standard meanings of red, yellow and green signal lights may not be modified by alternatives, at the risk of increasing instead of lessening the public hazard. So also purely administrative provisions, whether relating to records, to compensation for loading signs, to traffic experiments or to violation bureaus, have no place in the basic code but should be left to local ordinances. A national code to be effective must invite universal adoption, and above all must be simple. The accomplishment of the conference will be measured by the degree in which it shapes its code to these fundamentals.

New Light on an Old Problem

ONE OF THE civil engineer's classic problems, retaining-wall design, was carried forward into new dimensions and refinements a few years ago, when a wall of 170-ft. height and subject to quite unusual service conditions was built at the Fifteen-Mile Falls Dam on the Connecticut River. It was this undertaking that led to the epochmaking test-bin installation and experiments at the Massachusetts Institute of Technology, results of which were summarized in the series of articles by Dr. Charles Terzaghi which we were privileged to publish in recent issues. The new quantitative understanding of soil action upon walls which resulted from these investigations has its counterpart in the advancement of design procedure which utilized the experimental results. The considerations involved in the design, as presented this week by Dr. Terzaghi, are fully as instructive as the experimental results, and of perhaps greater interest; for after all the engineer's perennial task is to reduce a practical problem to such terms as will permit of so harmonizing it with scientific data as to approach a reasonable maximum of economy and efficiency. In the present case the difficulties presented by the action of earth alone and in conjunction with variable heads of water, both standing and percolating, justify the intricate studies and calculations presented. The large potential savings at issue in the design, as well as the ingenious use of permeable banks and tight core in the backfill of the wall to reduce hydrostatic pressures, increase the interest of the problem. Its methods of solution represent a step forward in retaining-wall history and throw the light of practical study on a subject that for many generations was regarded as a favorite field for the exercise of purely mathematical ingenuity.

Bridge Inventories

ANY BUSINESS can be operated better and more efficiently if it has information currently available on the condition of its plant. Private industry learned this long ago and gladly pays the cost of complete and accurate inventories. Highway departments, on the other hand, with a physical plant as valuable and varied as many great industrial plants, commonly treat the inventory question in haphazard manner. That this attitude may be a mistaken one is suggested by Ohio's experience with a record system that covers in detail every one of the 7,000 bridges on its 12,000-mile state highway system and some 57,000 culverts in addition. This exceptional record system has proved its worth in its three years of existence, on two occasions in quite spectacular manner, the first in 1930, when it facilitated the emergency task of constructing hundreds of small bridges in all parts of the state in order to spread employment, the other last year, when the records were of great value in apportioning federal money between primary and secondary road systems. In many other ways Ohio's experience has shown that currently accurate inventories are quite as useful for highway departments as for industrial plants.

City Planning

CAUGHT BETWEEN the extremes of present enthusiasm for initiating state planning and the zeal for tearing down isolated slum buildings, the rational study and programming for metropolitan development has been somewhat obscured. During the 1920-29 decade much constructive thinking was done and a fair amount of public education was effected in the development of city planning. Planning was aided during those years of urban real-estate speculation by the public desire for any information helpful in predicting future values. During the last few years, when metropolitan areas have been disintegrating faster than the rate of building, much of the incentive for planning was lost, though the need for it has grown very greatly. The result calls for a re-orienting of city planning. As reviewed on another page of this issue, the situation indicates that metropolitan development can be expected to continue in its present form without important change, but with emphasis on proper balance of city building. Slum clearance and low-cost housing projects require proper evaluation and correlation with the city planning program, and there must be adequate tie-in with state and other large-area planning. In all, city planning is more important than ever before. The experiences of the last few years make it the first agency of restoration and stabilization of values.

Chicago's Sewage Problem

HIGH-DEGREE TREATMENT of all the sewage of Chicago at an earlier date is the essential objective of the report rendered by the PWA special board of review on the Sanitary District, summarized elsewhere in this issue. To achieve this result, the board would let the existing

West Side Imhoff tank plant continue to operate without supplementary treatment of the effluent, and, in place of now providing a refining treatment here, spend the money in building the projected Southwest Side plant. Under this plan all the sewage of the city would be treated. It is logical that the board accepted high-type treatment as standard, for the local conditions make low-type treatment inadmissible. It is also logical to postpone to a later date a decision on what to do with the effluent from the West Side Imhoff installation until experiments on supplementary treatment can be made to find out how to salvage as much of the $16,000,000 investment as possible. If a moderate degree of purification is permissible, the salvage is possible. The board feels that a 70 per cent effluent would be satisfactory (and probably could be obtained through chemical treatment). It reasons that with the other activated-sludge plants in operation the weighted-average degree of purification would then be 85 per cent. This was considered adequate by Chief Justice Hughes when he was acting as master to the Supreme Court in 1929. If now the court will grant the district permission to go ahead on the recommended basis, work may proceed within a reasonable time.

To Revive Home-Building

FAILURE of the government's plan for large-scale low-cost housing is one of the great disappointments of the recovery campaign. To take its place, the Administration now proposes an ambitious home-modernizing and home-building campaign financed under government guarantee and strengthened by a fragmentary mortgage insurance and reform plan. It is declared to be a plan to revive construction, put the capital-goods industries back on their feet and absorb many of the unemployed. This is the program now relied upon to carry forward the recovery drive.

The shift of objective from low-income housing to the general dwelling field is surprising, to say the least. But whether or not the Administration was firm in its desire to provide for the bitter need of the exploited occupants of unhealthy and offensive tenements, there is known to be so great a need for dwelling improvement and construction that from the recovery viewpoint the shift of objective may be disregarded. It is certain that a serious dwelling shortage is approaching; virtual stoppage of house-building for more than five years has built up a deficit of at least two million family units, and the recently reported first results of the government's real-property survey support this figure.

Home-building thus affords a market of more than five billion dollars, a market temporarily dormant. Repair and modernization also present large opportunities, for neglect of dwelling maintenance and betterment has been so extensive that the estimate of 1 to 1½ billions of work needed is not immoderate. Do the measures contemplated by the Administration promise to utilize these markets effectively?

To stimulate modernization, home owners who need to make repairs or who desire to put in a sun porch or new plumbing are to be granted loans, to be repaid on the installment plan. The ghosts of the modernizing campaigns that were conducted here and there to boom local business are to be called back to life by the magic influence of installment selling—one of the devices most widely blamed for intensifying the pre-1929 inflation and the subsequent collapse. Have these old methods any virtue for the present purpose? Is it reasonable to expect that the struggling home owner will want to increase his investment, already far too burdensome, on government credit? Except for the landlord, for whom it is not intended, the installment modernizing plan can hardly have much appeal, and its recovery power appears negligible.

To stimulate home-building, government-guaranteed loans are to be made by private lending agencies at five per cent. The plan holds out a certain bargain to the prospective home builder, but it offers little incentive to the central element of the mechanism, the loaning agency. And it overlooks entirely the vital fact that home owning has largely become discredited; the average citizen has been thoroughly disillusionized by recent experiences with destroyed property values, dead investments and foreclosure threats. Home ownership is not likely to become widely popular again until the real-estate credit system is reclaimed from its medieval state.

In its short-time possibilities as an aid to business revival the new plan for economic salvation justifies only pessimistic views. The reforms in financing methods that are included would prove valuable in the long run were they intended to be permanent; a single large mortgage of long term and low interest with continuous amortization is a good beginning toward mortgage reform, and the associated insurance measures are helpful. But the plan as a whole is not put forward as a reform measure. Its purpose is to revive the capital-goods industries, and it is the only form of stimulation that now appears to be contemplated.

The obstacles in the way of capital-goods revival are well known. Commercial and industrial construction, as well as the improvements of utilities, must wait the renewal of actual business demand. And as the latter in turn is recognized as dependent on the capital-goods activities, there is the more imperative reason for centering effort on public works in the heavy construction field and on house-building in the lighter field. But if to the languid and unpurposeful attempt to vitalize public-works construction there is now added an unpromising house-building plan, the prospect is discouraging.

In the present home-repair and building scheme, fallacy is added to the failure of the low-cost housing program. It is fallacy to predicate a business revival plan on the cooperation of some of the very groups that successfully obstructed the earlier housing plans; to depend on a modernizing campaign based on emotional appeal; and to lure the bankrupt home debtor deeper into debt. Instead of seeking to produce employment through home-building, the national policies should seek to produce home-building through employment. The true way to stimulate both repair and building of dwellings is to give the average man—even the forgotten man—fair and steady employment, and by modernizing realty practices to make home-owning profitable and satisfying. These facts may as well be faced now as later. There is little gain in replacing a program that has failed with one based on fallacious optimism. The capital-goods industries deserve and demand something better than that. Business recovery itself depends upon it.

CURRENT NEWS

Roosevelt Requests $1,322,000,000 for Works and Relief

AN EMERGENCY relief fund appropriation of $1,322,000,000 has been requested of Congress by President Roosevelt "to take care of relief and of orderly re-employment at least until the early part of the calendar year 1935." From this fund $940,905,000 would be assignable for general relief and public works. The President has requested wide latitude in the administering of the fund, in accordance with needs, but the following amounts have been tentatively tagged for specific use:

Public Works	$500,000,000
Highway Construction	100,000,000
Naval Construction	40,000,000
Tennessee Valley Authority	48,000,000
Public Buildings	35,000,000
Inter-American Highway	5,000,000

The items in the above table of tentative allocations total $728,000,000, leaving a fund of about $213,000,000 available to the President to be spent for direct relief and work relief.

There is also $285,000,000 set aside for the continuation of the Civilian Conservation Corps out of the $1,322,000,000 fund.

Fund Completes Budget

The appropriation of $1,322,000,000 would complete the $3,166,000,000 emergency recovery fund for the fiscal year 1934-35 proposed by President Roosevelt in the budget of January. Previous appropriations made by Congress total $1,844,000,000 and the President limited the present appropriation request to keep within the total budget figure.

Of the balance of $1,322,000,000 the President's message to Congress enumerates deductable items totaling $96,095,000, which must be met, leaving $1,225,905,000 available for all relief purposes up to July 1, 1936.

Fund Held Sufficient

Commenting on the size of the appropriation when compared to factors of the need for relief money and the balancing of the federal budget, President Roosevelt said:

"In my judgment an appropriation in excess of the above amount would make more difficult, if not impossible, an actual balance of the budget in the fiscal year 1936, unless greatly increased taxes are provided.

"The present estimate should be sufficient as a whole to take care of emergencies of relief and of orderly re-employment at least until the early part of the calendar year 1935. If at that time conditions have not improved as much as we hope today the next Congress will be in session and will have full opportunity to act."

For providing work relief the President apparently favors a program of planned civil works. Indications point to a renewal of the CWA next winter on a program more carefully laid out in advance than the emergency operations of last winter.

Columbus Ordered to Proceed With $3,400,000 Sewage Plant

Construction of the proposed $3,400,000 activated sludge type sewage disposal plant at Columbus, Ohio, will go forward as a result of a state supreme court ruling. The supreme court ruled that the municipality is under orders of the state department of health to construct the sewage treatment plant and that such an order is not subject to public referendum. The city council, some weeks ago, had ordered the city service director to enter into a contract for the construction of this PWA financed development when referendum petitions were filed.

To avoid long court delay the question was taken directly to the supreme court contending that the referendum petition prevented the city from going ahead with the construction. The court decision was in agreement with the attorney-general's ruling that the order of the department of health was not subject to a referendum.

Marmet Dam on Kanawha River Placed in Operation

The Marmet Dam, which will increase the navigable depth of the Kanawha River from 6 to 9 ft. has been completed and is in full operation. This is the first of four navigation dams to be built on the Kanawha and the Ohio by the U. S. Engineers. The present structure is of the roller-gate type and is equipped with double locks each 56 ft. wide by 350 ft. long providing a lift of 27 ft. at normal river stage. The construction of this dam and the roller gates were described in detail (*E.N.R.*, September 21, 1933, p. 337).

House Votes $460,000,000 Bill for Federal Highway Aid

By a vote of 255 to 26 the House of Representatives approved the Cartwright bill authorizing $460,000,000 for federal highway aid. The legislation provides authorization, but does not carry an appropriation, and is not mandatory on the appropriations committee. The highway construction program provided, according to report, does not conflict with the President's public works program, as the amount which would be actually appropriated for the fiscal year, commencing July 1, would not exceed about $100,000,000.

The overwhelming majority given the bill in the House is considered an indication of the attitude of that body on public works legislation. An amendment was adopted, during the consideration of the bill, stipulating that 25 per cent of the state allotments should be applied to secondary and feeder roads. The bill includes $50,000,000 for roads, trails and bridges in national parks, forests and public land, and also $10,000,000 for the anticipated reconstruction of highways destroyed by floods, earthquakes or landslides in any state.

There were several suggested amendments to the bill which were defeated by large majorities. A proposal that the size of the fund authorized be cut in half was shouted down and another motion to refer the bill back to committee was defeated 207 to 85. A suggestion by a Massachusetts representative that the weight given to population, in determining state allotments, be increased from ½ to ¾, claiming that the heavily populated states would not benefit sufficiently by the measure, was defeated.

Housing Survey Shows Shortage Results From Doubling-Up During Depression

PRESSURE from housing shortage may be expected as the economic situation continues to clear and families now doubling up with friends or relatives strike out for themselves, according to information revealed in preliminary reports on the real property survey now under way by the Department of Commerce. The survey has finished all field work and has made public reports on about 20 cities out of the 63 investigated. All work is expected to be completed by the middle of July. The department is going over the information brought out by the survey and hopes to be able to point the way to new markets for business. The electrical division is compiling a study of electrical appliance needs in the cities covered by the survey. This will probably be followed shortly with a study of construction and plumbing and heating needs by other divisions of the department.

Reports on the cities for which data has already been assembled show that while there is considerable doubling up, or more than one family living in a dwelling unit, there are a number of towns where the per cent of existing vacancies in dwelling units is greater than the number of extra families housed in a single dwelling. The following table shows the difference between the percentage of doubling up and the percentage of existing dwelling vacancies in the cities for which data have so far been compiled in the survey:

	Families Doubled-up (Per Cent)	Existing Vacancies (Per Cent)
Casper, Wyo.	2.7	12.4
Columbia, S. C.	6.6	5.2
Butte, Mont.	4.7	15.9
Asheville, N. C.	11.1	11.3
Nashua, N. H.	7.8	5.1
Burlington, Vt.	5.6	5.1
Paducah, Ky.	11	5.7
Greensboro, N. C.	10.1	3.9
Boise, Ida.	5.9	4.9
Little Rock, Ark.	8.4	8.4
Wichita Falls, Tex.	7.8	8.3
Hagerstown, Md.	8.3	5.3
Sioux Falls, S. D.	5.5	3.2
Fargo, N. D.	3.3	6.4
Reno, Nev.	3.5	12.3
Frederick, Md.	8.1	2.4
Albuquerque, N. M.	3.5	7.6
Lincoln, Neb.	6.6	7.7
Pueblo, Col.	4.9	8.3
Phoenix, Ariz.	4.8	7.1

Home Building Program Outlined by President Roosevelt

THE LONG-AWAITED home renovation and construction program, to be sponsored by the Administration as a recovery aid for the building trades was outlined by President Roosevelt in a message to Congress on May 14. Calling attention to the increasing need for improvement in housing conditions, the President outlined a four-point program providing for federal insurance of repair and construction mortgages combined with provisions for issuing certificates for low-cost financing through building and loan associations. Following the reading of the message, identical bills were introduced immediately into the House and the Senate to carry out the program.

The essential points of the program provide for:

1. Long-term loans, up to $2,000, at low interest rates, by the federal government for the modernization of property, particularly residences.

2. Guarantees on new mortgages (at a 5 per cent interest rate) for existing homes up to 60 per cent of "currently appraised value," by the Federal government, with guarantee of mortgages on newly completed homes up to 80 per cent of value.

3. Organization of private mortgage associations, under federal supervision, to provide long-term, low-cost financing for home owners and builders.

4. Insurance of building and loan association shares and certificates.

President's message

Among the ideas expressed by President Roosevelt in sponsoring the home renovation program were the following:

"Many of our homes are in decadent condition and not fit for human habitation. They need repairing and modernizing to bring them up to the standard of the times. Many new homes now are needed to replace those not worth repairing. The protection of the health and safety of the people demands that this renovizing and building be done speedily.

"The Federal Government should take the initiative immediately to cooperate with private capital and industry in this real-property conservation. We must lay the groundwork for this effort before Congress adjourns its present session.

"The purpose of the program is twofold: First, to return many of the unemployed to full and gainful occupation; second, to produce tangible, useful wealth in a form for which there is great social and economic need.

"The modernization of the program will furnish national guidance and support for locally managed renovizing campaigns throughout the country and protection of home owners against unwarranted cost advances. . . . Modernization of commercial and industrial structures is envisioned, as well as residential, but the new features providing governmental assistance are confined largely to home improvements."

The bill creates a Home Credit Insurance Corporation, capitalized at $200,000,000 and empowered to issue obligations

(Continued on p. 649)

Mississippi River Flow Lowest During Spring Months Since 1861

Since September 1933 the gage readings of the Mississippi River at St. Louis have registered a record low for every month, compared to the record for the same months since 1861 when the government began to compile daily records. The unusually low flows result from the lack of precipitation in the entire upper Mississippi Valley. Although the Missouri River run-off has been subnormal, it is noticeably higher than the corresponding spring run-off on the Mississippi.

During the month of April, the river dropped to 3.2 ft. on the gage, the lowest on record for that month, corresponding to the previous record of 3.4 ft. in 1931. The mean reading for the month of April was 6.2 ft. while the normal mean for the month is 18.5, indicating the extreme decline in the 1934 spring run-off.

During the month of May, the river has dropped from 6.4 ft. on the first of the month to 3.4 ft. May 8. The normal average for the month of May has been 18.4 ft. and in 1933 the average reading was 20.4 ft. for the month.

Cleveland Approves Bonds For Operating Expenses

At a special election May 15 Cleveland voters decided, by a vote of 80,118 to 46,066 to issue deficiency bonds in the amount of $4,000,000 to supplement the city's operating revenues for the balance of 1934. These bonds will give the administration slightly more than $11,000,000 for operating expenses, compared to $13,000,000 spent for municipal services last year in Cleveland. The bonds are to run for fifteen years. Their complete financing, it is estimated, will amount to $6.48 per capita, spread over the life of the bonds.

The approval of this deficiency bond issue provides no permanent financing and will merely insure restricted operating services until conditions improve, taxes increase, or new sources of revenue are found. If the city is able to dispose of this $4,000,000 in bonds, and no difficulty is anticipated, such essential city services as garbage collection, fire and police protection and public health work will not have to be as drastically curtailed as was originally announced.

FIELD BUILDING IN CHICAGO PLACED IN SERVICE

The 43-story, $12,000,000 Field Building, the only large commercial structure built in Chicago during the past few years, was recently put into full service without any formal dedication ceremonies. It rises 535 ft. above La Salle St., covers 61,000 sq.ft. of ground and has 1,000,000 sq.ft. of rentable floor space. It is located on the site of the historic 11-story "first steel-skeleton skyscraper," Home Insurance Building, built 50 years ago. Graham, Anderson, Probst & White, Chicago, were the architects and George A. Fuller Company, New York, general contractor.

Roofing Chapter of Construction Code Is Approved

CHAPTER 7 of the construction code, covering the roofing and sheet metal division of the industry was approved by Administrator Johnson for the President on May 10 and becomes effective May 25. The construction work covered by this chapter is more inclusive than its title indicates. It includes all contracting operations calling for:

1) Furnishing and installing roofing materials of various kinds on roofs or as siding for buildings, or other structures where applicable, including new work, reconstruction or maintenance.
2) Supplying and laying plastic subflooring, but not including these types laid in portland cement grout.
3) Waterproofing any type of structure below or above ground surface except integral waterproofing.
4) Damp-proofing interior surfaces with any type of material which may be applied in fluid or semi-fluid state.
5) Calking door and window frames.
6) Fabricating and installing sheet metal work of No. 10 U. S. or lighter gage, or of coppersmith work regardless of gage required as part of a specific project on or in buildings and structures, except that which is a part of a product of another industry.

Divisional code authorities

Recognition has been given by the Administrator to divisional code authorities established under approved chapters, and national headquarters have been established for the following:

Chapter 5, divisional code authority for Cement-Gun Contractors, Inc., 649-650 Munsey Bldg., Washington, D. C., Arthur J. White, executive secretary.

Chapter 6, divisional code authority of the Electrical Contracting Division, 420 Lexington Ave., New York City, Lawrence W. Davis, executive officer.

The organization meeting of the Construction Appeals Board will be held in Washington, May 18, at which time a chairman and secretary will be elected, and the board organized to carry on its duties under the code.

Area agreements

The hearing on the proposed area agreement affecting masonry contractors in New York City and part of Long Island reconvened in Washington on May 16 to receive any further information from those who previously appeared at the hearing on May 1. There has been considerable apprehension expressed in the industry that public hearings might be called in Washington rather than in the territory where the agreement would be effective. While no official announcement has been made, it is understood that the hearing on this first proposed agreement was held in Washington while the general procedure for area agreements was being developed, and that the policy will probably be to hold such hearings, hereafter, in the home areas.

W. W. Brush to Retire from New York Water Department

William W. Brush, chief engineer of the Department of Water Supply, Gas and Electricity of New York City, is to retire from active service at the end of May, following nearly 40 years of service in the water departments of New York and Brooklyn. Mr. Brush has been chief engineer of the department since January, 1927. He is the second engineer connected with the water services of the city to retire in less than a year's time, Thaddeus Merriman, chief engineer of the Board of Water Supply, having retired in December, also following nearly 30 years of service with the city. The Department of Water Supply, Gas and Electricity is the city's operating department and the Board of Water Supply has charge of the planning and construction of new supplies, chiefly construction of the Catskill system.

William W. Brush

Mr. Brush is 60 years old, a native of Orange, N. J., a graduate of New York University in the class of 1893, receiving his civil engineering degree the following year and his master's degree in 1895. He entered the service of the city of Brooklyn as a chainman in the bureau of water supply in October, 1894, becoming an assistant engineer in 1897, and continuing in the same position following the consolidation of Brooklyn and New York, in which work he was on survey and design and subsequently on the laying and relaying of distribution mains, the laying of the high-pressure fire service mains, and the construction of two filter plants. In February, 1907, Mr. Brush went to the Board of Water Supply as an assistant engineer on the design of the main gate house for the Ashokan reservoir. Subsequently he became department engineer in charge of field and office studies for the general design of the delivery system within the city limits for the Catskill supply. In 1910 Mr. Brush returned to the operating department as chief engineer for Brooklyn of the Department of Water Supply, Gas and Electricity, and subsequently became deputy chief engineer of the department under the late Merritt H. Smith.

PWA Allots Fund Of $3,830,000 for New York Hospital

A LOAN and grant of $3,830,000 has been allotted to New York City for the construction of an eight-story unit at the Bellevue Hospital, contingent on the city submitting a satisfactory report showing a balanced budget. The request for an original loan and grant of $4,400,000 was reduced to the smaller figure, according to Administrator Ickes, through a change in plans. The change in plans will not delay the start of construction on the project which is programmed to employ an average of 400 men for an eighteen months' period. The building will be about 340 ft. long by 40 ft. wide, for an eight-story height with a south wing about 100 ft. by 80 ft., and two north wings about 112 ft. by 32 ft.

Two non-federal allotments were made during the past week according to the report of the Administrator. A direct grant of $8,000 was made to the school board of Wetumka, Okla., to cover 30 per cent of the cost of a one-story school building. The other allotment was a loan of $36,000 to the Missouri Southern Railroad Co., a short line located in Missouri, to be used for the purchase of a new diesel locomotive.

More loans reduced

Loans and grants to four more non-federal projects have been reduced to direct grants of 30 per cent at the request of the recipients who are now able to finance the balance of the project in the private market. These four changes in allotment have resulted in releasing $319,000 which has already been reallotted to projects on the waiting list. A total of 193 changes have been made to date resulting in the release of $23,840,000 for reallotment to projects on the PWA waiting list.

The loan and grant of $159,000 to Fort Scott, Kansas, for school building construction has been reduced to a grant of $45,500, and the loan and grant of $170,000 to Milwaukee County, Wis., for improvement to the sewer system has been reduced to a direct grant of $50,000. The other two changes in financing were of smaller size.

A grant of $4,800 to Manchester, Ga., has been changed to a loan and grant of $18,000 which will finance the construction of an extension to the sewer system and build a new sewage treatment plant.

New railroad loans

Loans to three railroad companies totalling $2,493,000 have been announced by Administrator Ickes. The Grand Trunk Western Co. received a loan of $250,000 for the purchase of rails to be laid on a 20 mile stretch of track near Cassopolis, Mich.

The Delaware & Lackawanna received a loan of $1,043,000 which will be used to convert 20 road engines into switching engines and recondition almost a thousand wooden box cars. This work will be done in the company's shops at Scranton, Pa., and will result in recalling about 200 shop men. The Chicago Great Western Railroad received a loan of $1,200,000 for the purchase of 500 steel box cars.

Roosevelt Outlines Home Building Plan

(Continued from p. 647)

(guaranteed as to principal and interest by the government) when needed to rediscount paper representing credits for repair and modernization, and to insure mortgages on new homes up to 80 per cent of the original principal obligation and new mortgages on existing homes up to 60 per cent of the appraised value.

National mortgage associations would be chartered by the federal government with a minimum paid-in-capital of $5,000,000 to be privately subscribed as prerequisite to lending on or dealing in mortgages insured by the Home Credit Insurance Corporation and to borrow for that purpose up to a fixed ratio of their outstanding capital stock. The associations would be controlled and supervised by the Federal Home Loan Bank Board.

A Federal Savings & Loan Insurance Corporation, managed by trustees who are members of the Federal Home Loan Bank Board, would be created to insure the shares and certificates of building and loan associations. In the event of default by an insured institution, the insured account, not over $2,500, would be paid 10 per cent in cash, 50 per cent of the remainder within one year and the balance within 3 years. The corporation is authorized to serve as liquidating agent for defaulted institutions, to issue bonds and borrow money.

Home Loan Bank enlarged

The new bill amends the Federal Home Loan Bank Act to enlarge the discount facilities of the federal home loan banks, and to empower them to advance funds to member institutions for the purpose of relending to home owners for repair and modernization. The home loan banks also would be authorized to issue joint obligation bonds and debentures without the necessity of collateral trusts. The Federal Reserve Act would be amended (1) to remove the present limitation of a 5-year maturity and a 50 per cent of appraised value on home mortgage loans by national banks when such mortgage loans are insured by the Home Credit Insurance Corporation; (2) to classify as commercial loans, 6-month loans on residential construction and to make any such loan eligible for rediscount by Federal Reserve Banks.

All-American Canal Bid Involves 20,000,000 cu. yd. of Excavation

Bids have been called for excavating about 30 miles of the main All-American canal which will convey waters of the Colorado River to the irrigation development in Imperial Valley without crossing into Mexican territory. This federal project, financed by the PWA, will be constructed by the Bureau of Reclamation; the total cost is estimated at $27,000,000.

The present specifications divide the project into seven schedules, each schedule involving the excavation of from five to seven million cubic yards of common excavation, except schedule No. 7, which consists of about 500,000 cu.yd. of rock work. Bids will be received on any or all of the seven schedules from any bidder and will be opened by the Bureau of Reclamation at Yuma, Ariz., June 7.

SOCIETY CALENDAR

AMERICAN SOCIETY FOR TESTING MATERIALS, Atlantic City, N. J., June 25-29, 1934.

AMERICAN SOCIETY OF CIVIL ENGINEERS, annual convention, Vancouver, B. C., Canada, July 11-14.

AMERICAN WATER WORKS ASSOCIATION, annual convention, New York, June 4-8, 1934.

NATIONAL CONFERENCE ON STREET AND HIGHWAY SAFETY will hold its fourth annual meeting, Washington, D.C., May 23-25.

INTERNATIONAL CITY MANAGERS' ASSOCIATION, annual conference, St. Louis, Oct. 15-17.

SMOKE PREVENTION ASSOCIATION, annual convention, June 19-22, Buffalo, N. Y.

PENNSYLVANIA SEWAGE WORKS ASSOCIATION will hold its annual meeting at State College, Pa., June 27-29.

Houston Municipal Bond Sales Spread to Obtain Better Prices

With the sale of $1,425,000 of Houston, Tex., general revenue bonds, scheduled for May 14, and a June election called for voting a $2,500,000 issue of general waterworks bonds, there has been some apprehension that these two large security offerings coming close together would tend to reduce the premium which city officials expect to realize on the first offering. As a result, the city comptroller has secured a promise from the Council to withhold the $2,500,000 waterworks bonds, if voted, from the market, until September 15. This action is expected to result in a premium about $5,000 higher than would be obtained if the two bond sales had conflicted.

Court Halts Letting of Contract for PWA-Financed Power Plant

Reversing a lower court decision, the Kentucky Court of Appeals has entered an order restraining Middlesboro, Ky., from letting contracts for the construction of a municipal power plant and distribution system. The proposed development had been allotted $328,000 by the PWA. Litigation attacking the contracts for the loan with the Federal government for the PWA allotment, was instituted by taxpayers and the Kentucky Utilities Co., which has been supplying power to the community for some time.

The court held that the ordinance permitting the borrowing of Federal funds is subject to referendum since 58 per cent of the voters have signed a referendum petition.

The suit claimed that the city is adequately served by three lines from the utility's plant at Pineville, Ky. As a development in the controversy, the city owes the company $35,000 for municipal services, according to report, and the company has withheld $3,000 of municipal taxes.

New Jersey Law Permits Bayonne Port Development

A $10,000,000 port terminal development at Bayonne, N. J., is authorized by a law passed by the state legislature. The legislation was opposed by several existing New Jersey port developments. The bill will permit the state board of commerce and navigation to sell to the Central District, Inc., for $1,500,000 riparian rights to about 500 acres of submerged land.

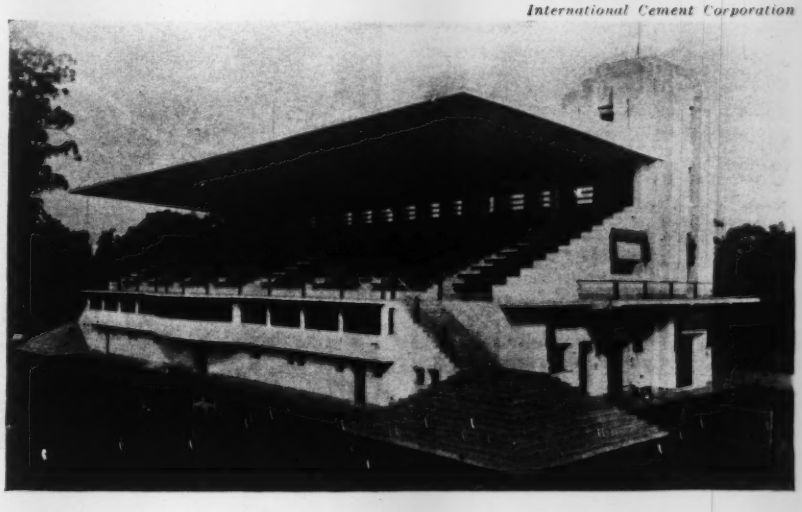
CONCRETE CANTILEVER ROOF SHELTERS GRANDSTAND

The new reinforced concrete race-track grandstand of the Commissariat of the Hippodrome at La Plata, Argentina, is structurally notable because of a cantilever roof having an overhang of 44 ft. The roof, 131x65 ft. in area, is composed of slabs generally 3 in. thick although over the column supports the thickness is 7⅞ in. The slabs are rectangular, supported by cantilever beams 20x43 in. in section on 5-ft. centers and transverse secondary beams, carried on the cantilever beams. The whole roof was placed in 42 hr. of continuous pour, using rapid-hardening cement in a 1:2:3 mix. There are no expansion joints in the roof.

The observation gallery at the front is for the use of the club authorities. The towers at either end contain the water tanks. The interior of the grandstand houses the changing quarters for the jockeys and race committees of the club.

Engineers for the structure were T. Marfort and J. Szelagowski.

International Cement Corporation



Mexican Interests Will Build 75-Mile Railroad in Durango

Plans submitted by the Durango state government, the National Railways of Mexico and several Mexican mining and lumber companies for the construction of a railroad 75 miles long from Villa de Vicente Guerrero to San Juan del Mezquital, southern Durango, have been approved by President Rodriguez. The work will cost about $560,000 and efforts are being made to commence the construction before summer, according to General Carlos Real, governor of Durango. The railroad will serve a rich mining and agricultural district which lacks adequate transportation facilities. The National Railways are especially interested in the plan, and, according to reports, will advance most of the money.

Personals

J. E. TEAL has been appointed Transportation Engineer for the Chesapeake & Ohio Railway Co. with offices at Richmond, Va.

ARTHUR B. CULMER has been appointed head of the city engineering and water departments, Bellingham, Wash. He succeeds ARTHUR EDSON, who recently resigned as city engineer.

L. H. HORNSBY has been appointed assistant engineer of bridges for the Seaboard Air Line Railway with headquarters at Norfolk, Va. Mr. Hornsby has been in the service of the company for a period of 12 years and received his technical education at McGill University.

WALTER L. COUSE and WILLIAM K. SAUNDERS, both formerly with the W. E. Wood Co., Detroit, Mich., have formed a partnership of Couse & Saunders, Engineers and Contractors, Detroit. Prior to their service with the Wood company Mr. Couse was with the public works department at Flint, Mich., and Mr. Saunders was in the Detroit water board office.

JOHN A. KNIGHTON has been appointed chief engineer of the department of plant and structures for New York City. Mr. Knighton has been in the department for 30 years and was the engineer in charge of construction for the Queensboro Bridge. He is a graduate of Cornell University.

ALFRED M. LUND, examining engineer for the PWA, has been assigned as resident engineer inspector for the levee project of the St. Francis Levee Board at West Memphis, Ark.

CONSTRUCTION STATISTICS OF THE WEEK

ENGINEERING AWARDS for this week total $23,355,000. This is very close to the average week to date and 19 per cent improvement over last week. Public works awards this week are 67 per cent higher than last week while private awards are down 60 per cent. State and municipal contracts account for the public works awards since federal awards are down slightly below last week and are less than half the average week to date.

Highway contracts recoverd this week with a $9,922,000 total compared with $4,348,000 last week and public buildings, totalling $5,241,000, are well ahead of last week's total of $3,229,000. Commercial building awards dropped back to their average volume after the high total last week. All other classes of work were lower this week.

Large awards include the Dobbs Ferry school, New York, $800,000; the State Armory at Baltimore, $750,000; the Greenwich, Conn., high school addition, $581,000; the shops and hangar for the U. S. Air Corps at Middleton, Pa., $663,000; highway lettings, $805,000 by Alabama, $651,000 by Kentucky, $893,000 by Texas, $522,000 by Indiana and $1,145,000 by Missouri. Lexington, Ky., awarded Contr. 3, storm sewers, $572,000; Illinois State Highway Department awarded $904,000 in bridge contracts and Los Angeles the Figuroa St. Tunnel No. 4 at $385,000.

New capital is low this week, state and municipal bond sales are the lowest since the last week of March, $3,760,000. Adjustments in PWA allotments netted a decrease of $441,000 in cumulated allotments to date.

CONTRACTS

(Thousands of Dollars)

	Weekly Average May 1933	Prev. 4 Weeks	Week May 17 1934
Federal Government	$751	$5,429	$2,016
State and municipal	14,360	15,384	18,862
Total public	$15,111	$20,813	$21,022
Total private	4,438	4,448	2,333
Week's total	$19,549	$25,261	$23,355
Cumulative to date: 1933	$333,630	1934	$472,405

NEW PRODUCTIVE CAPITAL

(Thousands of Dollars)

1934	Week May 17	Cumulative
State and municipal	$3,760	$116,209
PWA allotments, S&M	—487	186,837
Corporate issue	..	13,975
PWA allotments, private	36	50,417
Total, Non-Federal	$3,309	$367,438
PWA allotments Federal Constr.	..	49,436
Total new capital	$3,309	$416,874
Cumulative to date: 1933	$113,377	1934 $367,438
Rescinded or reduced allotments (from 1933 PWA totals)		$43,323

NOTE: These figures include private bonds, and stocks sold for productive purposes; state and municipal bonds for construction; PWA loans and grants to states and municipalities, including the special highway funds; PWA private loans, and allotments for Federal construction.

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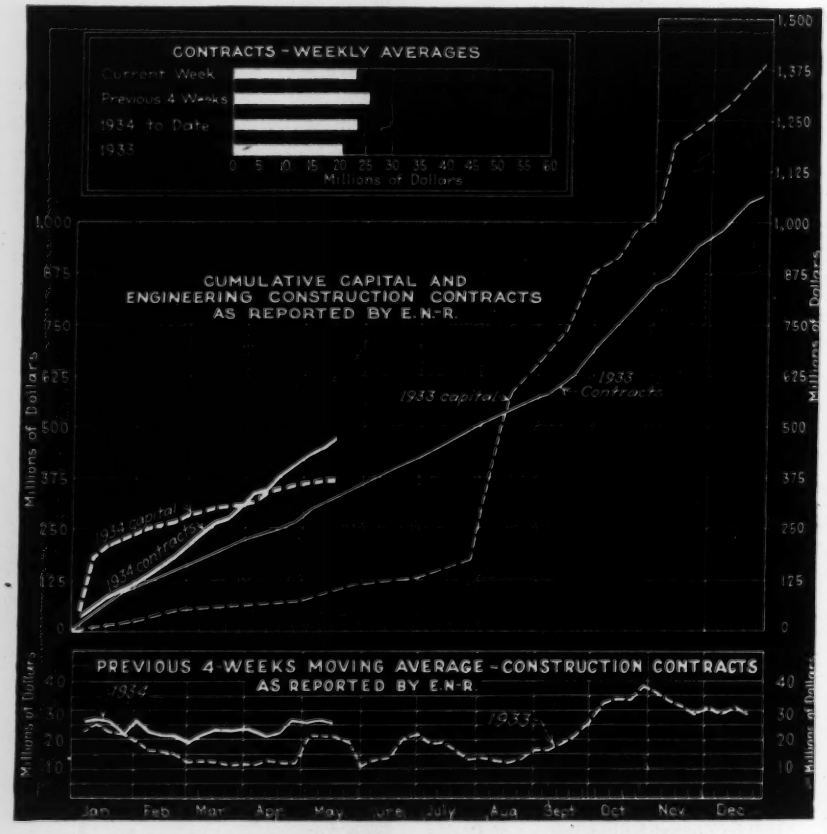
E.N.-R.-Cost		E.N.-R.-Volume	
May, 1934	199.61	April, 1934	109
April, 1934	195.86	March, 1934	108
May, 1933	164.39	April, 1933	65
1933 (Average)	170.18	1933 (Average)	102
1932 (Average)	156.97	1932 (Average)	127
1931 (Average)	181.35	1931 (Average)	220
1913 (Average)	100